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CABLEWAYS.

By SPENCER MILLER, Assoc. Am. Soc. C. E.

READ JANUARY 17TH, 1894.

The cableway may be defined as a hoisting and conveying device employing a suspended cable as a trackway, and to such the author will limit this paper. Cableways are sometimes called "cable derricks" or "cable hoists." What are known as "wire rope tramways," "wireways," "ropeways," whether inclined or otherwise, have no capacity for hoisting, and are limited to the sole function of conveying.

While wire rope tramways date back to the early part of this century, the cableway finds its birth practically in an inclined hoisting and conveying device invented about 1860, by Charles Shuman, and the device as he designed it at that time is still in extensive use in the slate quarries of Pennsylvania and Vermont.

These cableways consist of a winding engine with one drum, a suspended cable, a cable carriage traveling on the cable track-way, a fall block adapted to rise and fall from the cable carriage, and a hoisting rope operating the same. Fig. 1 shows the early form of incline

cableway with the cable carriage in gate. The angle of incline of the cable is about 23° from the horizontal (see Fig. 2), measuring to the air line from top of "*A*" frame to tail anchorage. The hoist rope

FIG. 1.

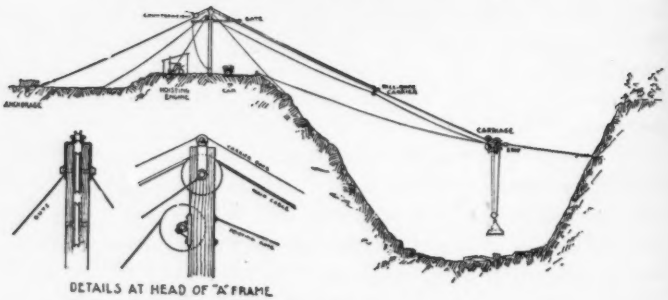

FIG. 2.

is multiplied three times from the carriage down to the load, the result being that at the minimum angle of 23° the resistance to drawing the

carriage up the incline is greater than the effort required to hoist the load. Therefore the carriage, resting against a stop on the cable, will not begin to ascend until the fall block reaches and comes into contact with it. Means are provided that the fall block may collide with the carriage without injury to any of the parts.

After unloading the fall block is raised to the carriage, the carriage drawn up the incline sufficiently to release the gate, then the hoist rope is paid out, the carriage descends by gravity down the incline cable until it reaches a stop on the cable, and, continuing to pay out hoisting rope, the fall block descends to the pit.

The wheel shown in the head of the "*A*" frame, or tower (Fig. 1), is one of the early mistakes, as illustrated by the repaired cable to the right side of the head of the frame. The best practice now is to employ an oak saddle, grooved with an arc of large radius to correspond with the stiffness of the rope (see Fig. 3).

The cableways shown are introduced about 30 ft. apart. There are probably 300 of them, known as "Blondins," now in existence. The load usually carried is from 3 to 5 tons. Sometimes single masts have been used in place of "*A*" frames. They, of course, require more guying for their stability.

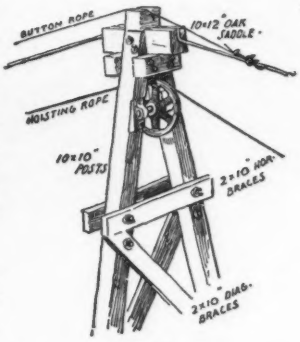

FIG. 3.

As the span of these cableways becomes greater, the necessity of supporting the hoisting rope between the tower and carriage becomes manifest. Fig. 2 shows the earliest device of its kind. This fall-rope carrier follows the carriage down the incline, drawing out a small manilla rope, permitting the carrier to go down only half way. At the other end of the manilla rope a counterweight is secured, so that on the return of the carriage and fall-rope carrier, this piece of manilla rope could be drawn out of the way from entangling with the other ropes. This carrier is limited to short spans; not more than two have ever been used, and it is not to be considered as a solution of the difficulty. Much experimenting has been done to improve it.

Horizontal cableways of short span were used in the construction

of the piers of the St. Louis bridge, and a patent, dated January 24th, 1871, was granted to Henry Flad and James B. Eads, the former a Past President and the latter a Past Vice-President of this Society. The span was short, and no fall-rope carriers were required.

About 1884, Milo W. Locke, a contractor, designed an incline cableway for building the bridge piers of the Baltimore and Ohio bridge over the Brandywine, near Wilmington, Del. He afterwards took this same plant, making it horizontal by the addition of an endless rope for the horizontal motion of the carriage, and installed it in a quarry near by, of which he became manager. On this plant he used the chain-connected fall-rope carriers which consisted of a series of blocks with 8-in. or 10-in. wheels, to run on the main cable, spaced about every 50 ft., connected with ¼-in. chains. These heavy and cumbersome fall-rope carriers were the source of endless annoyance. The span of the original cableway was about 750 ft. Mr. Locke received patents about 1884 for his invention. The cableways employed at the Sodom Dam and Tilly Foster Mine were modeled after this.

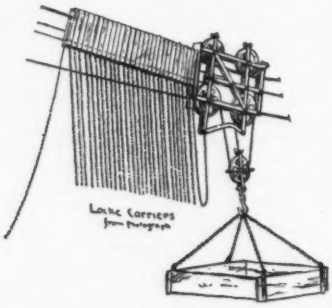

FIG. 4.

At the Sodom Dam 32 chain-connected carriers (Fig. 4) about 20 ft. apart were used (the span being 667 ft.). The hoisting rope does not need supporting oftener than every 100 ft., but with the chain-connected carriers, the chains themselves must be supported; hence the great number of carriers employed.

The weight of chain and carriers used on this plant was about two tons. The chains swing about and get entangled in the fall block, and with each other, they limit the speed, and wear the cable.

This Sodom Dam cableway was not designed by an engineer, and possibly to this fact is traceable the deplorable accident in October, 1889, when the main cable broke, resulting in the death of a laborer. The reason it broke was because it was strained up too tight, allowing too small a sag or deflection. This cableway was an assemblage of parts picked up here and there, without regard to efficiency. The engine was built in New York, the carriage in Pennsylvania, the cables in New Jersey. The main cable broke with a sag of 15 ft., and a load of 6 tons. With a span of 667 ft,, sag 15 ft., and load of 6 tons, the strain on a 2-in. cable is about 80 tons. This cable, when new, tested and broke at 132 tons. It is not surprising, therefore, that a cable used constantly for nearly two years, should break at 80 tons, when its ultimate strength, new, was only 132 tons, showing a factor of safety less than two. Cableways designed by the writer employ a factor of safety from four to six.

The towers on this plant should have been high enough to allow a sag of 33 ft. However, the plant was repaired, the towers were raised, so as to allow a sag of 22 ft., and the dam was completed with no further accident. The Sodom Dam was fully described before the Society on April 5th, 1893, by Walter McCullough, Assoc. M. Am. Soc. C. E.

When the Tilly Foster mines were begun on the open-pit plan, two horizontal cableways were employed, with chain-connected fall-rope carriers, and, as soon as the depth permitted, these were changed to incline cableways, employing the same fall-rope carriers. This is the first instance where cableways were used for "open pit mining," and to Mr. Clinton Stephens, contractor, belongs great credit for this novel installation; 329 000 cu. yds. of rock were thus moved to uncover 600 000 tons of ore.

One great advantage of using a cableway lies in the employment of a shallow skip, which, as will be observed, is easily filled by hand, with no preliminary work after a blast. At least two tons per man per day more may be filled into a skip 18 ins. high than into a car 36 ins. high.

Long and high-speed cableways were not practicable with the chain-connected fall-rope carriers. Fig 5 shows the first departure therefrom, and represents the beginning of a development in the line of improved fall-rope carriers. As will be observed, an auxiliary rope, about $\frac{5}{8}$ in. diameter, is suspended above the main cable, held in a par-

allel position to the main cable, by passing under wheels in the cable carriage.

On this rope a series of buttons are secured, whose diameter increases with the distance from the head tower. Slots in the head of the carriers, corresponding to the diameter of the buttons, allow each of the carriers in passing down the incline to be stopped at its proper button. These carriers have small wheels to roll on the auxiliary or button rope. Thus, the heavy cumbersome chains are dispensed with, and these fall-rope carriers, spaced by buttons, and weighing about 100 lbs., answer all the requirements of chain-connected carriers, weighing, with the chain, 1 000 lbs.

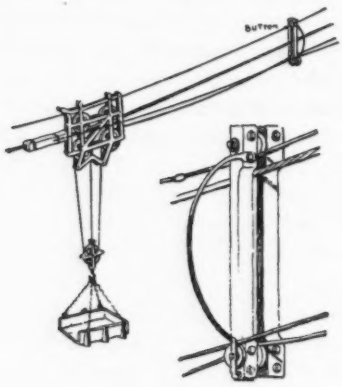

FIG. 5.

The button stop fall-rope carrier was next applied to a horizontal cableway of 855-ft. span, at Ogden, N. J. Means had to be provided for drawing the fall-rope carrier out with the carriage, as gravity was not to be depended upon as in the last case. To this end, two horns were placed upon the carriage, which served the twofold purpose of lifting the carriers bodily from the cable, thus dispensing with wheels on which the carrier might run on the main cable, and also, on these horns the carriers were taken when distributing them along the cable.

The original carrier was of wood and iron. The development of a few years has brought the carriage and carriers up to a standard form, as shown in Fig. 6. The cable carriage is usually built with two main

cable wheels, but in some instances three wheels have been used, notably in the Point Pleasant plant, 1 505 ft. span, which will be described later. The various ropes are indicated in the illustration. The horn in front of the carriage picks up the carriers as it passes toward the engine and also carries them out as the carriage recedes; the buttons on the button rope take the carriers from the horn and leave them spaced along the main cable at proper intervals for supporting

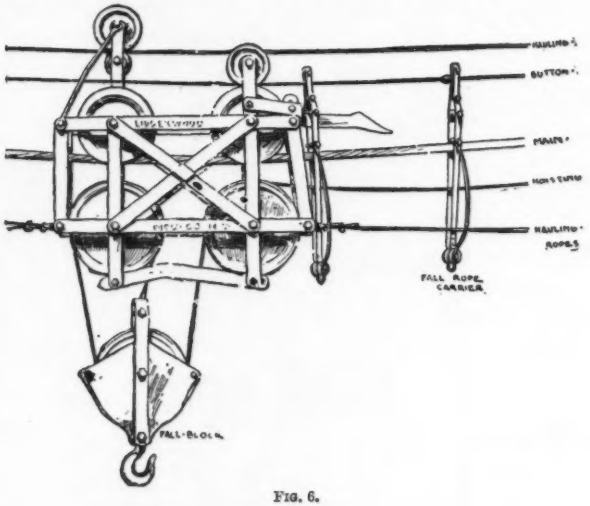

FIG. 6.

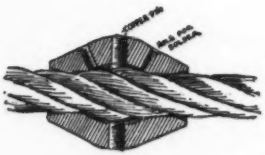

FIG. 7.

the hoisting rope. These buttons (Fig. 7) increase in size, receding from the head tower, as also do the corresponding slots in the head or top of the carrier.

Fig. 8 illustrates an engine with double cylinders with cranks connected at an angle of 90°, and is fitted with reversible link motion. The drums are 54 ins. diameter, regular friction type, one to carry the

hoisting rope, and the other is turned with a curved surface, as shown, and carries the endless rope. The endless rope is wrapped around the drum five or more times, enough to secure sufficient friction to keep it

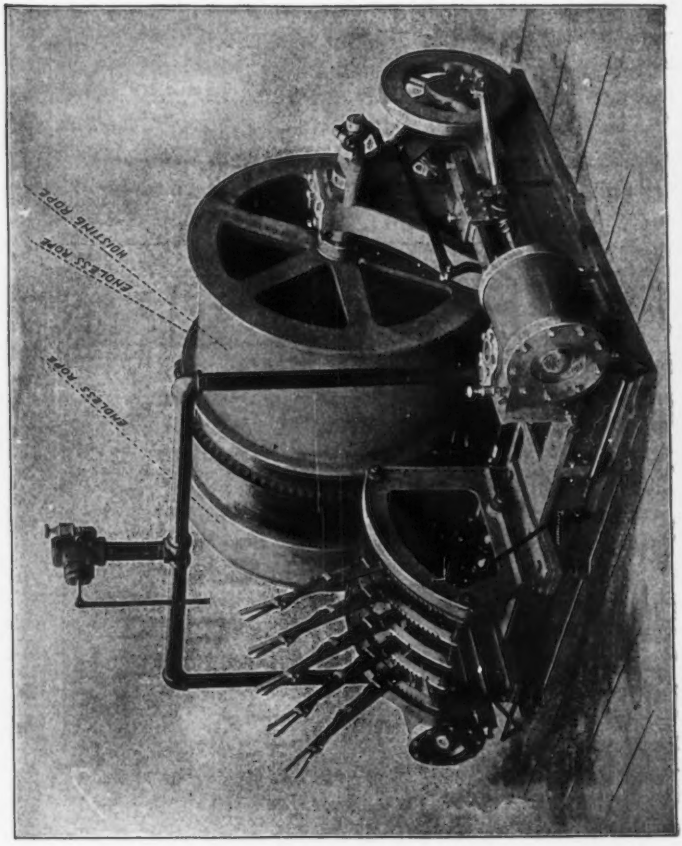

FIG. 8.

from slipping in the opposite direction to that in which the drum is turning, and the ends are passed over the sheave wheel on the towers and made fast to the front and rear wheels of the cable carriage.

The hoisting drum is independent of the other, and, being of the

same diameter, winds at the same rate of speed, and keeps the load at the same height, if so desired. This drum has a band brake by means of which the load can be sustained. The reversing lever, friction and brake levers, are all brought to a central position, so that the operator can work all of them in one position.

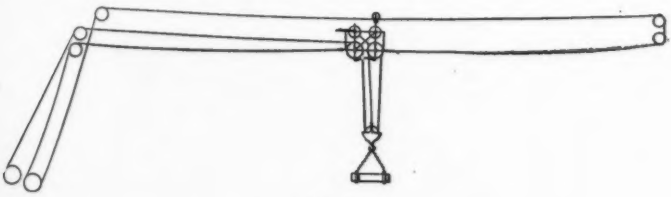

FIG. 9.

In Fig. 9 the method of leading the ropes on the long-span horizontal cableway is shown. The endless rope serves to hold the carriage in place while the hoist is being made. For horizontal motion the hoist rope is drawn in and the endless rope is payed out at the same rate of speed. As will be observed, the load may be hoisted or lowered at any point under the line of the cable, and the horizontal motion is given to the load at any height to which it may be raised.

The cableway at the Austin dam is thus described by E. W. Groves, Engineer in Charge of Construction.

"From the natural conformation of the ground this was an ideal place for the location of this piece of machinery. The west bluff rises nearly vertical to a height of 65 ft. above the crest of the dam, while the east bluff rises 10 ft. above the dam. This cable is suspended on two towers; the easterly one, in which is situated the hoisting engine, being 70 ft. high, and the westerly one 30 ft. high. The main cable on which the carriage moves is 2½ ins. in diameter, 1 850 ft. long, and 1 350 ft. between points of support. The load carried is about 6 tons.

"On the west side of the river and 1 200 ft. up stream from the dam is situated a limestone quarry, from which extended a gravity railroad along the west bluff directly under the cable. A car load of rock could be run from the quarry to the cable, taken up at once, and transported to derricks on any portion of the work. A double track was used near the dam, so that an empty car was ready to receive the empty skip, and a loaded car was also ready when the carriage came for a load.

"Under the cable at the easterly end of the dam extends the railroad bringing granite from the quarries, which can be taken from the cars

and transported to any portion of the work in the same manner as the limestone. Mortar is mixed on the wall near the workmen, sand and cement being taken out by the cable. Loads of about 5 tons are usually carried.

"This cableway is capable of supplying about 180 cu. yds. of material per day, where the distance to be traveled is not more than 1 000 ft."

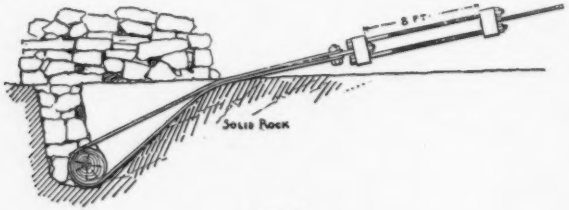

FIG. 10.

The heavy take-up device (Fig. 10) is not intended to be used in erecting the plant, but it is to take up the stretch occurring as the cable is used, or in raising the cable as the dams increase in height.

A great advantage of the cableway system is that when the plant is put in place it need not be moved or altered until the work is done. The frequent changes of position required where an ordinary contractor's railroad is used for moving material on such works increases largely the expense and time required. A criticism which might be made on the Austin plant is that many of the stones have to be handled twice, the cableway transferring them to derricks. This could have been avoided by the use of several paralleled cables, the load being taken out on whichever one was nearest to the place where it was to be deposited. A second advantage of the cableway is found in the fact that it is entirely clear of the work itself, and is safe from injury by flood or fire, and is reasonably safe from explosions from the work below.

The importance of this has been fully shown at Austin. No less than eight different times during a year has work on the dam been interrupted by high water, and at one time the water was 43½ ft. above low water-mark. Had the common plan of extending a railway out over the dam been adopted these delays would have been greatly lengthened by the time necessary to replace the portions of track and trestle-work washed away at these points.

Plate XLVIII shows the cableway used for building the Basin Creek

PLATE XLVIII.
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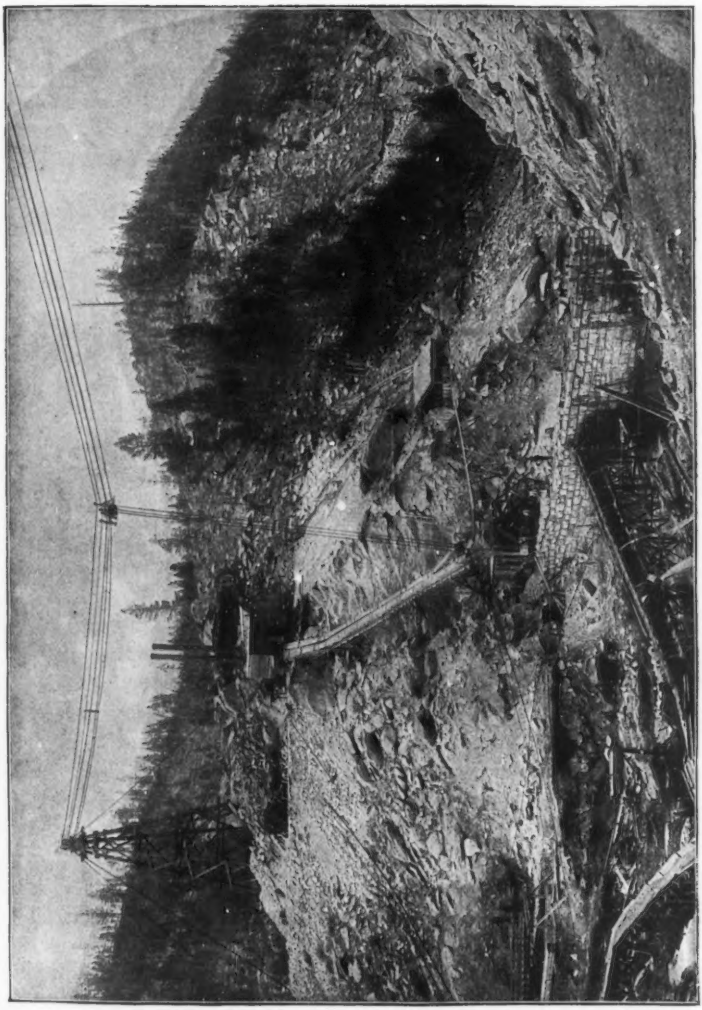


Dam for the water works of Butte, Mont. The dam is 120 ft. high, 300 ft. long, and was designed by Chester B. Davis, M. Am. Soc. C. E. The cableway spanned the dam and quarry 892 ft. between towers, one 80 ft. high, and the other 15 ft. high. Stones were taken direct from the quarry, and placed in the dam without further handling, and by the use of a snubbing post (Fig. 11) and horse all the stones were laid, no derricks being required or used.

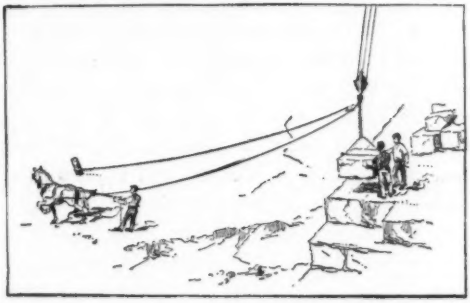


FIG. 11.

The cement mixer used on this work was moved by the cableway when it was necessary to put in a new foundation, requiring but a fraction of the time that would be required to do it by hand.

During sixteen days in June 1 430 cu. yds. of masonry were laid with an average of 86 men; this included six masons, quarrymen, firemen, and all labor employed about the dam and camp. This is an average of nearly 15 cu. yds. of finished work per day per mason.

The U. S. Corps of Engineers, under Lieutenant W. E. Craighill, used a cableway for building the lock and dam on the Coosa River, near Lincoln, Ala. The plant was erected in the spring of 1891; span, 1 000 ft.; main cable, 2¼ ins.; load, 8 tons.

The stone for this lock and dam was procured from a quarry about two miles back from the site of the dam; a 3-ft. gauge road was built and equipped with locomotives and cars. Each car held as its body a skip which was picked off the car by the cableway and by it delivered into place. The material used was brick and loose rock.

Plate XLIX shows a general view of the cableway at Point Pleasant, W. Va., taken when the water was very high. It shows to the right the coffer-dam surrounding the walls of the lock. The dam will lie

directly under the line of the cable. The distant tower is 100 ft. high. To the left is the head tower, located, as will be noted, near the quarry. The stone here is quarried and swung around underneath the line of the cable, and transported to the stone-yards shown in the center of the picture.

The contractors say they expect to use the cableway for work on the river 125 days each year, and work will last at least five years. The cableway will be used every day for transporting stone from the quarry to the stone-yard. The supplies and materials arriving by boat are unloaded upon the coffer-dam or upon the dock boat, and brought ashore by the cableway. It is found a great convenience in taking men across the river, as it is much more expeditious than ferrying them.

This plant has the distinction of being the largest hoisting cableway in existence, the clear span being 1 505.5 ft., main cable, 2½ ins. diameter; maximum net load handled, 4 tons. A great deal of material is also brought in over a railroad siding on the other bank of the river. A seam of coal underlies the quarry and an amount sufficient for all purposes is mined and transported directly by cableway to the dredges, hoisting engines and pumps in the river as well as to the boiler and electric-light plant.

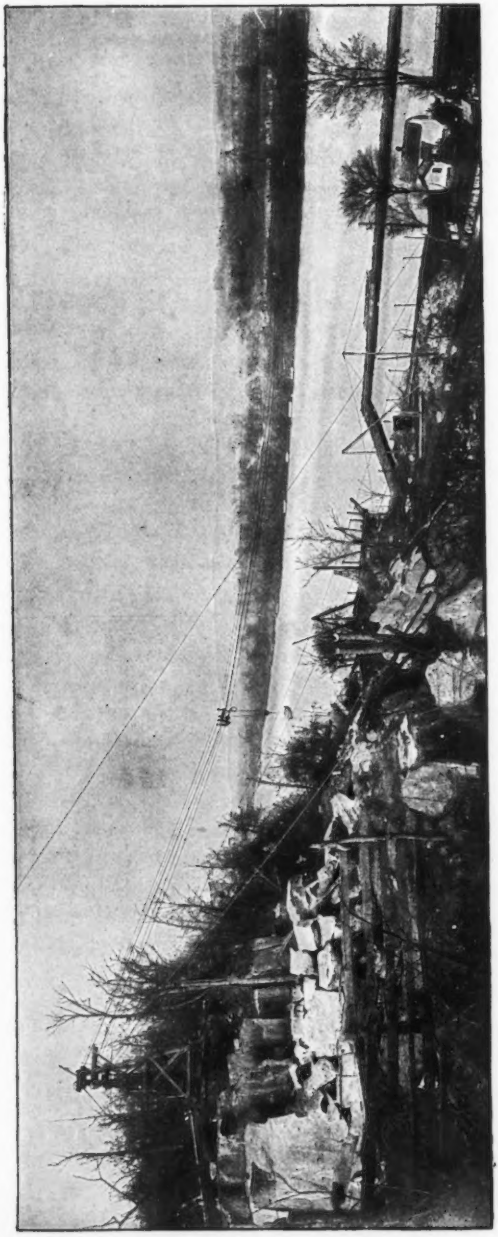
At Rochester, N. Y., two cableways were introduced side by side, and about 60 ft. apart, for the erection of a bridge, span 630 ft. and towers 50 ft. high. A 30 H. P. 8½ × 10-in. hoisting engine was used for each cableway. A system of electric signal bells was used for communicating with the engineer.

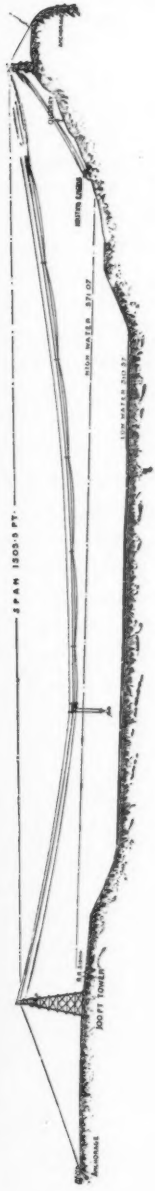
J. Y. McClintock, City Surveyor, furnished the following details of the operation and uses of this cableway.

The first stone was laid April 1st, and the last October 7th, 1893.

The estimated quantities for the bridge, all of which were handled by the cableways, were as follows :

Earth excavation	2 200 cu. yds.
Rock excavation	4 700 cu. yds.
Lumber in centers	350 000 ft. B. M.
Masonry in piers and arches	2 200 cu. yds.
Arch stones	2 278 cu. yds.
Concrete for spandril backing	2 660 cu. yds.
Coping	868 lin. ft.
Gravel	3 400 cu. yds.
Iron work	310 000 lbs.




SPAN 1303-3 FT.
300 FT TOWER

In the iron bridge over the canal the floor beams are 40 ft. long, and weigh several tons. These were put into place by taking hold of each end by a carrier and working the two simultaneously. Stones were laid both in the piers and arches, not only under each cableway, but midway between the two; that is, 30 ft. distant at right angles from either. This was done by hitching the hoist from each cableway to the stone, and hoisting on one while paying out on the other, thus dropping the stone into exact position. This feature was extremely advantageous to the contractors in laying all of the heavy skewback stones on the last piers, when the high water had carried away the derrick frame which was used in laying the other piers, and as the water still kept high, it seemed unadvisable to build another frame for the few yards of stone remaining to be laid.

To lay the piers a frame was used which straddled the piers and on the top of which a traveler was employed. The cableways were used to bring the stone to the piers, and it was carried sideways by these travelers. After a pier was completed, this whole apparatus was taken hold of by both cableways and in 10 minutes was set over on the site of another pier. This was such a simple matter that in some cases where the stones intended for a given pier were not at hand the traveler was moved to another pier, and when the stone on hand was used it was moved back to finish the first pier. The wooden centers for the arches were made very heavy, and were framed on a horizontal platform and lifted into place. For this work also the cableways were found very handy.

On account of the limited space available, the cableways were used for pulling the old piers out and removing 5 000 cu. yds. of stone blasted from the bed of the river. They carried stone to the crusher and then delivered concrete to all points. They deposited the spandril filling, laid the spandril walls and coping, carried the curb and paving stones for the street service, and, finally, dragged out the immense mass of timbers used in the centers.

In Baltimore, L. B. McCabe & Company, contractors, are now building, with the assistance of a cableway, the stone arch, North Avenue Bridge, over the tracks of the Baltimore and Lehigh Railroad, and the Baltimore and Ohio Belt Line.

The span is about 800 feet, and the maximum load handled 4 tons (Plates L and LI). The view, Plate LI, is looking towards the head

tower, and shows an early stage of the work. Several lines of street cars run on North Avenue, and, while the work of construction is going on, traffic is carried on over the temporary bridge, shown at the left.

All the stone required in the piers is transported from cars near the head-tower over the tracks and cars, and deposited near the pier; derricks then place the stones.

The contractor had great difficulty in securing permission to place the tower in the street on account of obstructing traffic, and it is due to the same objection that two or even three cableways were not installed, so that derricks could be entirely dispensed with.

That the cableway is considered to be a safe device for transporting material is clearly evidenced by these views, showing the cableway spanning several lines of steam and electric roads upon which there is a large daily traffic.

The Baltimore Belt Railroad through the heart of Baltimore (Ryan & McDonald, contractors), was constructed in part by opening the street and building the side walls, one at a time. This was successfully done by cableways which were employed because they offered but little obstruction to street travel.

A description of this tunnel* says:

"An interesting feature of the work was the use of cableways in constructing about 1 200 ft. of the south end of the tunnel near Camden Station. This section was constructed by open cut in the manner to be described. As the street carries a very heavy horse-car and wagon traffic at this point it was impossible to open up the street the entire width necessary for the tunnel. Side trenches were therefore excavated and the side walls of the tunnel put in. These being finished the street over the tunnel was replaced by a timber structure, as fast as the material was excavated.

"It was in excavating the side trenches that cableways were used, placed directly over the trenches. They were used in covering both the excavated material and the stone, bricks, etc., for the masonry, it being found that stone could be set by the cableway without the use of the usual derrick. The advantages to be gained by the use of cableways in this case are obvious when it is considered that no hindrance is opposed to the street traffic, which can be carried on directly underneath the apparatus."

Recent improvements in buckets have developed a self-filling scoop bucket which, drawn up the slope of the bank, gravel, sand or coal,

* *Engineering News*, May 18th, 1893.

PLATE L.
TRANS. AM. SOC. CIV. ENGRS.
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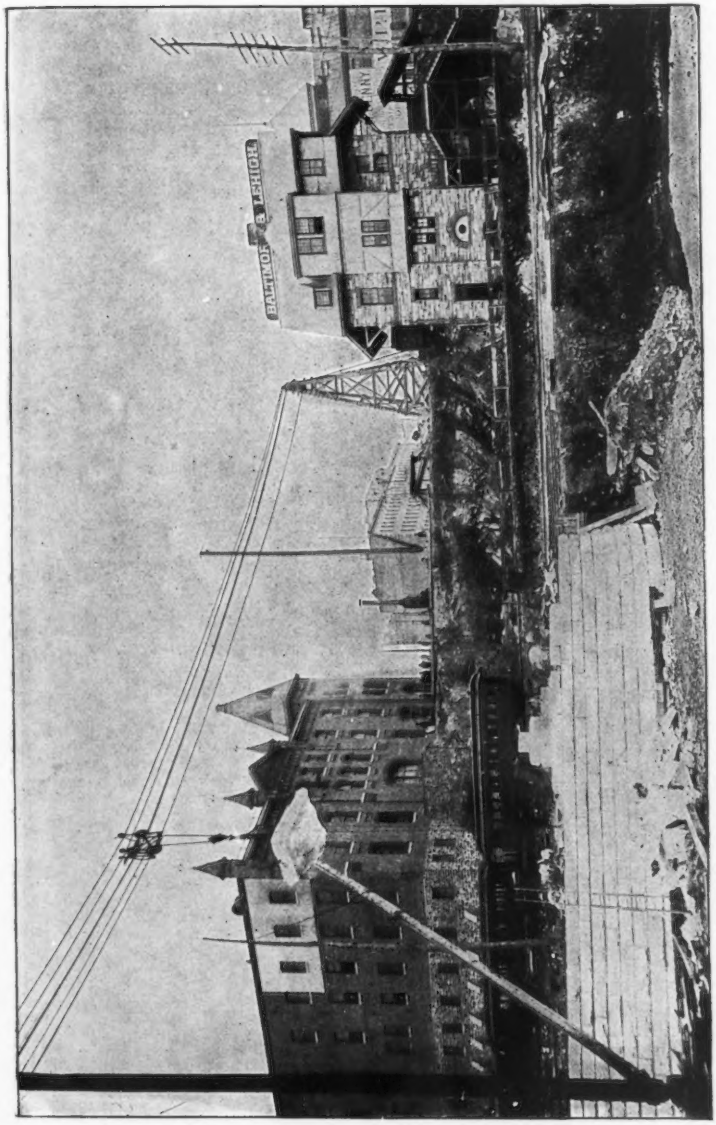
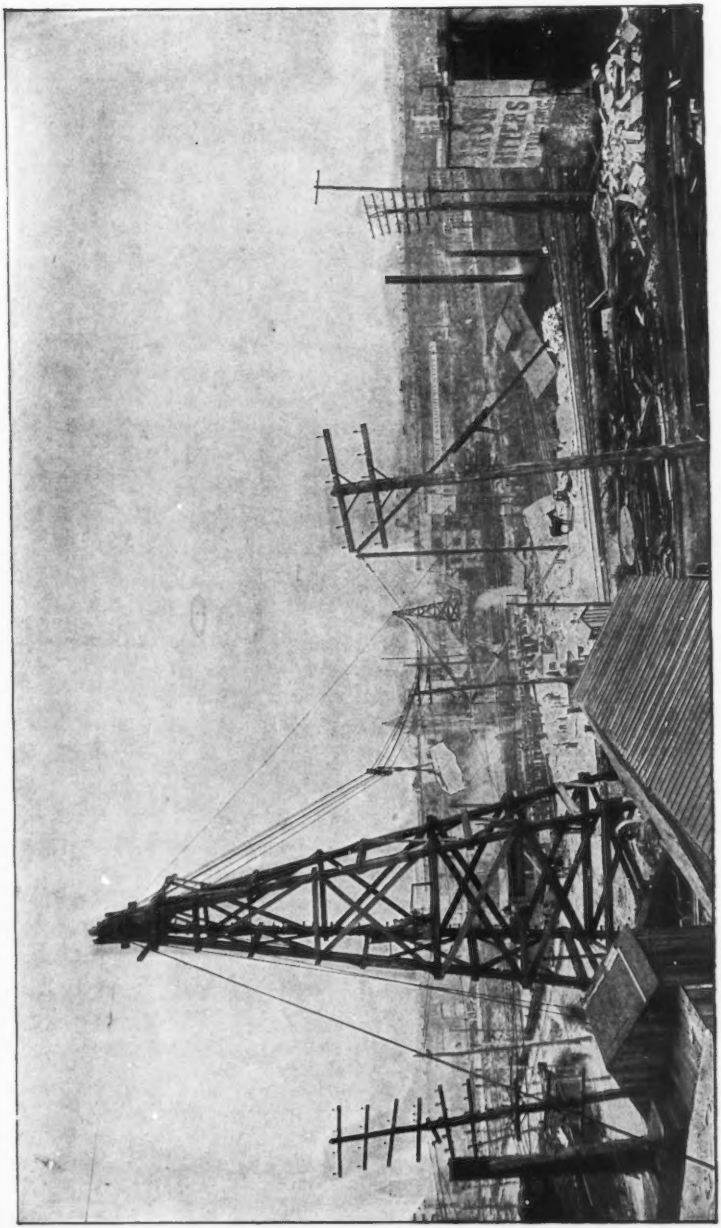




will fill perfectly. The bucket is then conveyed along the cable and automatically dumped into a car or hopper. The buckets hold from one to two yards, and a load is taken every one and a half minutes, being conveyed 300 to 400 ft., and automatically dumped.

The cableway is quite extensively used in logging operations, and is known as the "skidder," or skidding cable-way. In its use a tree, located some 10 or 12 ft. from the railroad track, is allowed to remain unfelled. To this tree chains and blocks are attached, and it becomes the "head" of the cableway. The span is usually 750 ft., the main cable from 1 to 1½ ins. Everything must be extremely light, because the moving must be done cheaply and quickly. This device does not pick up the load in the air entirely, but simply picks up one end of the log, or several logs, by tongs in the end of a single rope, and snakes them in with the other end of logs resting on the ground. They are simply raised high enough to clear obstruction.

The system calls for an auxiliary cableway, known as the "loading line." This has a block which is held practically stationary whereby by the use of an additional rope the log is picked up (after being delivered by the main cable) by the tongs, and delivered directly on the cars.

The time required after hooking on three logs up to the time they are delivered 700 ft. distant is less than a minute, and the sound of these logs rushing and crashing through the underbrush is enough to terrify an amateur.

A convenient form of cableway for logging in cypress is to use the mast of a scow as the head of the cableway. In this application of the system the logs are dragged in through the circle of 1 000 or 1 500 ft., and simply delivered into the water, where they are afterwards floated down to the saw mill.

The skidder is really the solution of logging in cypress swamps and where any other system would be impracticable. In its use for logging purposes, tramroads are placed in parallel lines about every 1 200 ft. apart through the woods. The skidder then works each side. The men operating it frequently stand up to the middle in the water and mud.

Traveling Cableways on the Drainage Canal near Chicago.—Portable or traveling cableways were invented some forty-three years ago by M. Pluchet, a Frenchman, and patent drawings show his portable

cableways used for building a canal. Whether it was ever put into use or not the writer is unable to state. The traveling cableway has developed many interesting things in its use.

Plate LII shows the traveling cableway, of which, at the present time, three are in use in the work at the Chicago Drainage Canal.

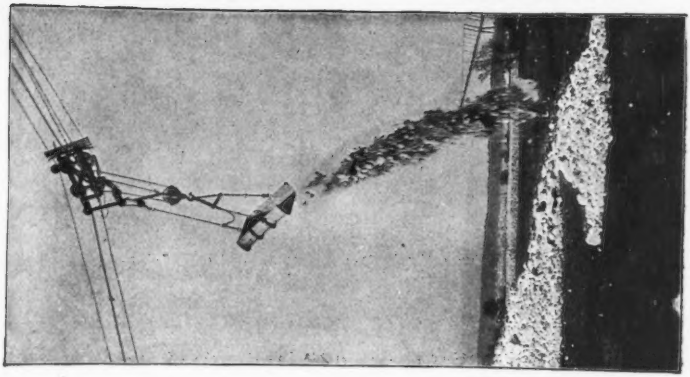
The span of this cableway is 623 ft.; the towers are 80 ft. high, and are located on cars 102 ft. long; each one of these cars rests on twenty-seven 33-in. wheels, and on this car or platform are located at the head tower the engine, boiler and the required amount of ballast, and on the tail tower simply sufficient amount of ballast for its stability.

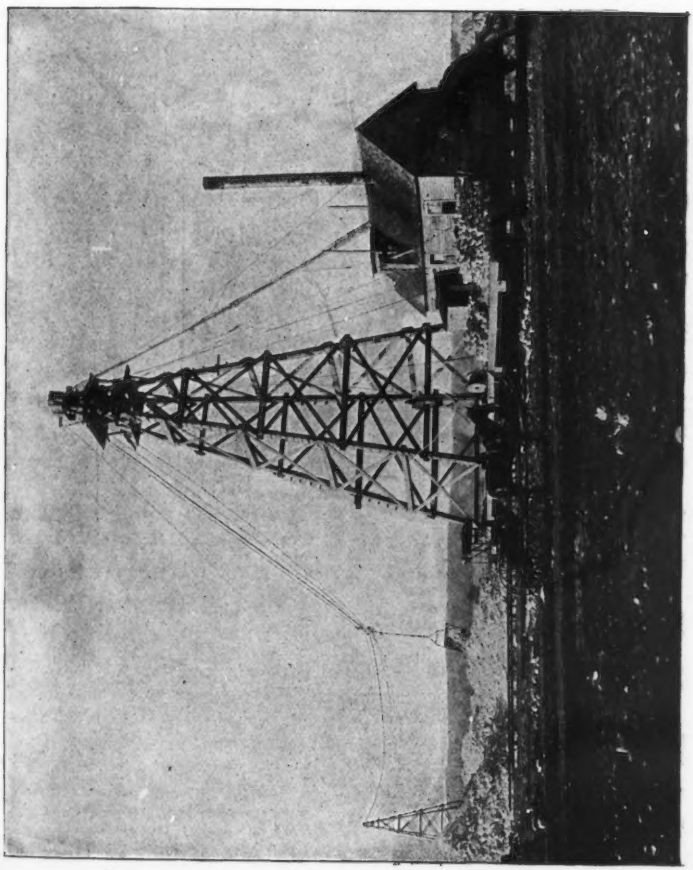
The main cable is 2 ins.; the engine is double 10 x 12, 50 H. P., with double drums.

The cableway is used merely to take material from the canal and and deposit it in spoil bank on one side.

The work is usually done on a breast 12 ft. high. The record shows that 32 men at the breast, filling skips and chaining large stones, average 14 cu. yds. per man per day, or 28 tons of material. Of course, it would not exceed 11 yds. if it were not for the fact that a great saving of time is accomplished by the handling of large stones out of the pit by simply chaining them to the fall block. Stones weighing 6 or 7 tons have been handled out of the canal in two or three minutes, thereby making a distinct saving in cost for drilling, powder for blasting and the cost of sledging; and the mere act of throwing the chain around a heavy block is only a fraction of the cost of filling the skip. On the first 12 ft. of the canal the stone broke up fine under the blast; on the second and third lifts, it breaks up so large that the work seems to peculiarly fit the cableway, and a large saving is made on all material that is handled out in large blocks.

In handling large stones the danger of slipping a chain or breaking a link is minimized on account of the fact that the stone may be handled horizontally at any height desired, just sufficient to clear the surface of the ground if required. The usual practice is to deliver these heavy blocks on the canal slope of the spoil bank, and they are frequently taken out at the noon hour with only two or three men at work, on extra time, so that when the men come in to fill the skips they can find nothing but uniformly broken material to handle. Many contracts were taken along the canal for channeling, blasting, hoisting,





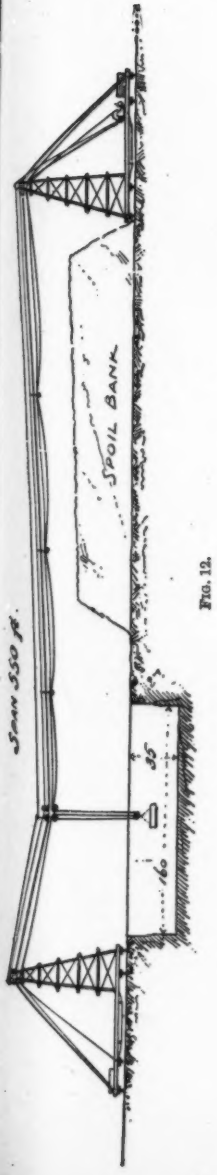


FIG. 12.

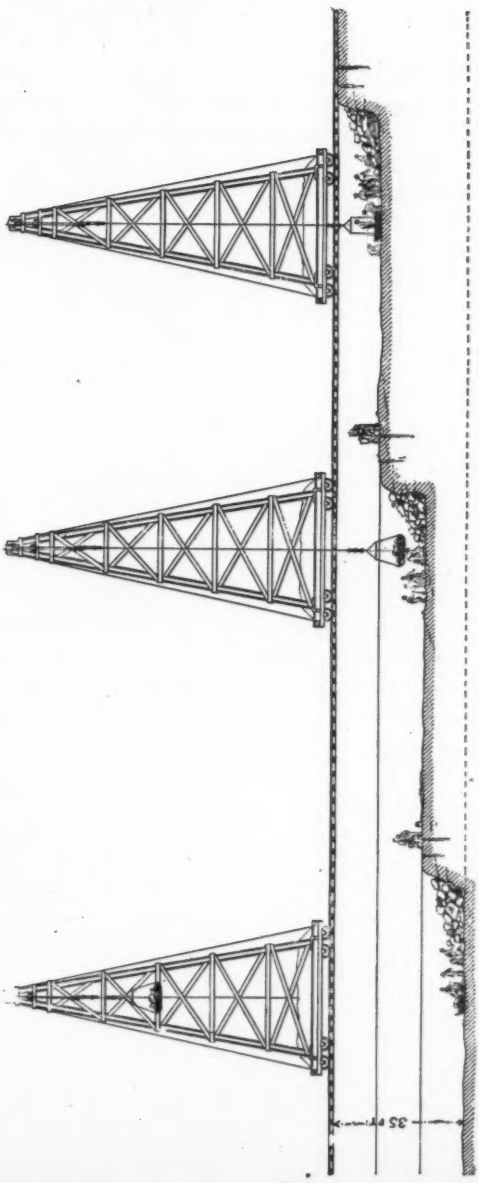


FIG. 13.

conveying and delivering the material to the spoil bank for 80 cents per cubic yard, working down to a depth of 35 ft., and 160 ft. wide. The facts are that probably not all of the contractors at this price are making money, but, nevertheless, it is certainly not to be disputed that on the sections that are properly worked, with the proper organization and proper machinery, so that the machines are constantly used to their maximum capacity, there is money to be made at 80 cents per yard, and such contractors working at this price are satisfied to take more work at the same price.

The aerial dump (Plate LII) has recently been developed which has increased the capacity of the cableway enormously. The idea is not new, but the peculiar application, however, is novel. The auxiliary rope arranged to draw up the bucket or skip is installed, passing from the carriage to the engine, parallel to the hoist rope, and winds on the same drum as the hoist rope. Therefore, when it is desired to dump the load it is merely required to draw in this rope at a little higher speed than the hoist rope, thus the material delivers without delay to the cableway. The instant the load has been spilled from the skip the carriage returns to the canal. For obvious reasons, any authentic records are difficult to obtain, but the following are understood to be correct. The record for the month of November was 230 skips daily. The greatest day's run was 258 skip loads of broken stone, averaging 2¼ yds. each, and 12 large stones averaging 2 yds. making a total capacity of 572 yds., or 1 144 tons. A few days ago the writer received a report that this cableway handled 302 skips in 10 hours, or 600 yds. per day; 500 yds. seems, however, to be about the average.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

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702.

(Vol. XXXI.—April, 1894.)

AN INSTRUCTIVE EYE-BAR TEST.

By A. C. CUNNINGHAM, Assoc. M. Am. Soc. C. E.

READ MARCH 21ST, 1894.

WITH DISCUSSION.

From a lot of nine steel eye-bars one was to be selected for test.

The bars were apparently perfect with the exception of one, and in this bar one of the finished heads contained three seams. It was decided to take this bar for testing, as the seams condemned it for use, and in the event of failure on account of the seams, it could be reheaded and retested.

The nominal size of the bar was 6 x $1\frac{9}{16}$ ins.

An elastic limit of 35 400 lbs. per square inch was developed, and the bar broke in the head, as indicated at *F* in Fig. 1, at a stress of 41 500 lbs. per square inch.

The fracture was square and granular, and started in a seam, as indicated at *S*.

The body of the bar gave no elongation or reduction.

The excess of area on the line *AB* over the body of the bar was 50 per cent.

The broken head was cut from the bar and enough of the body to reach sound material, after which the bar was reheaded and reannealed. An examination showed that the new head contained seams, quite a decided one being at the point *S*, Fig. 2, which was chipped out and found to be about $\frac{1}{8}$ in. deep.

At a stress of 60 200 lbs. per square inch a piece broke out of the new head, as indicated in Fig. 2.

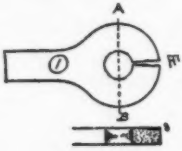


FIG. 1.

FIG. 2.

The fracture was square and granular, and in solid material, without visible flaws or seams.

On the second test, the bar gave an elongation of 7% in the body.

The specimen test representing this bar had given results as follows: Elastic limit = 36 600; ultimate strength = 63 100; elongation in 8 ins. = 30%; reduction of area = 49 per cent.

The object of full-sized eye-bar tests is to check the quality of the workmanship, the condition, quality and grade of the metal.

In this case the specimen test representing the bar indicates a a metal of the proper grade and of excellent quality. Was the workmanship and treatment of the bar defective, or is the specimen test misleading?

Even if the workmanship and treatment were defective on the first head, when the bar was reheaded especially for testing, it may be assumed that the best of care would be taken, and yet the failure was worse than in the first test.

What, then, is wrong? The following analysis of drillings taken from the face of the second fracture, Fig. 2, shows the cause of the failure: Carbon = .21%; phosphorus = .118%; manganese = .60%; sulphur = .126 per cent.

The phosphorus is high, nearly .02% higher than the limit which is set for structural steel, but probably not higher than in at least 30% of the material which is being made and used at the present time.

The primary cause of the failure is found in the sulphur, which is

over twice as great as the recognized limit for material which is not to be reheated, and over three times as great as the limit for forgings, with which eye-bars will class.

The probabilities are that this eye-bar was from the extreme top of an ingot where the impurities due to segregation are in excess.

With the knowledge of this high sulphur in mind, we can surmise what the trouble was with this bar. The sulphur at the end was probably just at the critical point beyond which the steel would have crumbled in rolling. In making the head on the bar, a small additional amount of sulphur may have been absorbed in the heating, and the severe nature of the work in forming the head was sufficient to severely injure the steel.

The behavior of this bar under test is a strong plea for the cropping of ingots. If the upper quarter or third of the ingot was discarded, the highly segregated portion would be removed, and the remaining portion would be fairly uniform, or at all events not dangerously segregated, as the top of an ingot is most likely to be.

The cropping of ingots has always been advocated for the removing of piping and unsoundness. With the knowledge of segregation which has been generally diffused during the past few years, the additional need of cropping seems almost imperative.

DISCUSSION.

PERCIVAL ROBERTS, Jr., M. Am. Soc. C. E. (by letter).—Before being able to draw a final conclusion, we should have more definite information as to the following points:

Was the specimen tested cut from the same bar from which the full-sized eye-bar which was tested was made?

What was the analysis of the specimen tested?

Were analyses made from different parts of the full-sized bar to determine the matter of whether the analysis of the broken head gave only local results, or were these results obtained also at other points in the bar?

Were any analyses or tests made from other bars made from the same melt as the bar in question?

Was the steel Open Hearth or Bessemer?

What were the conditions existing during the annealing of the bar? Also, what process was used, the character of the furnace, and the

nature of the fuel. Also, length of time the annealing took place, and how long afterwards before the bar was removed from the annealing furnace?

The character of this steel was such that it would be an extremely sensitive material to the effects of annealing, which should have been very thorough to produce beneficial results.

Mr. Cunningham's suggestions as to the cropping of ingots would cause such an increase in the cost of material as I fear not many customers would be willing to pay. Hitherto, the Government alone, for ordnance purposes, has enjoyed such a luxury as this. I do not believe it is necessary for structural work, as, by the use of a proper quality of material and by proper manipulation, no such segregation should take place as is noted in the present instance; although, I much question whether the instance here referred to is segregation, unless we have additional information or answers to the questions I have above asked.

By keeping the metalloids in the steel sufficiently low there is little practical danger of segregation which amounts to anything taking place. The extended use of Bessemer steel in this country has caused the use of material entirely too high in phosphorus for eye-bar purposes. I believe in no instance the phosphorus should exceed .08, but better .06, while sulphur should not be over .06. With these metalloids at these points and carbon about .16 or under there would be very little danger of segregation.

James Christie, M. Am. Soc. C. E. (by letter).—The analysis of the steel in the eye-bar head showed phosphorus and sulphur higher than is considered acceptable, but it is not uncommon to get satisfactory tensile tests from steel of similar composition. If these elements were found to be less in the opposite or unbroken head it would be safer to assert that the fault was due to segregation, and that the analysis did not represent the general character of the steel.

Porosity of the ingots, or cracks developed by blooming, might account for the seams in the defective head.

It will be observed that at the second test, the body of the bar must have been strained to the verge of rupture; the small difference in unit tension between the full bar and the test piece is a frequent result, possibly due to annealing.

I do not question the advantage of liberal top cropping of ingots to remove unsound or segregated material. Upset eye-bars are liable to fail in the head if not properly annealed. The steel should be uniformly heated to a temperature of about 650° Cent., and uniformly cooled.

R. H. Thurston, M. Am. Soc. C. E.—I imagine that the writer of the very pithy paper before us has stated the real cause of the defective quality of the eye-bar examined and tested. The cause of failure

and that of the seams described are probably identical. The phosphorus is certainly too high, and the total of hardening elements altogether too great; but the peculiarity of the analysis is in its large percentage of sulphur. Not only does sulphur cause "hot-shortness" by its volatility and tendency to diffuse itself at a welding heat, and by producing, when hot, a resultant stress throughout the mass of material affected by it; but it also gives decided tendency, I imagine, to the production of crystals throughout the whole of the metal thus affected. Very possibly this may account both for the brittleness and the crystalline structure of the steel; for it is well known that overheating is very apt, especially in metal containing sulphur, to produce just such crystals. In this case we have excess of sulphur, as proven by analysis, and overworking, with probably overheating, or at least too much work and too much heat for this particular metal, and their natural results in brittle steel and overstrained and defective heads.

I have an idea that such metal as this could be much more safely worked by the hydraulic forging machine—where it must be used at all—than by any system of hammering or rolling. I found the press system of forging in use, many years ago, in Vienna, where Mr. Haswell, the English manager of the State Railways, was employing it in making all sorts of running parts of machinery and rolling stock, and I was so much impressed with it that I introduced some account of it into my report to the Government on the machinery and manufactures of the time, and especially of the Vienna Exposition of 1873. But a much more complete account of what was then already accomplished is given in Blake's report, in the same volume, on Iron and Steel. The slow and steady flow produced by the press has a vastly different effect upon the metal than the quick, and often comparatively concentrated and superficial, action of a hammer, particularly if the hammer is too light for its work; and the encouragement by engineers of the substitution of the press for the hammer will, I am inclined to think, render available for structural and other work some qualities of metal that, to-day, must be condemned. I doubt if there is any serious objection to sulphur in quantities that are not too great to permit working the steel. In fact, it is said that, in the manufacture of ordnance, the Swedes have introduced small doses of that element to give toughness. I doubt if its presence, however, is to be considered an advantage, ordinarily; but it would be an advantage, could ores and irons and steels containing moderate doses of that substance be brought into use safely.

The avoidance, meantime, of such materials; the careful cropping of the ingot regardless of the magnitude of the wasted part; and the adoption of better methods of working are continually making us safer against the accidental introduction of such metal into structures

As illustrating the fact that simple heating, without heavy strain or

loading, will sometimes produce this crystallization or granulation, a sample bit of wrought-iron bar in my collection has interest. It is a piece of 2-in. bar, which once formed part of the grate under a gas retort. It was always hot, never what would ordinarily be called overheated, and was turned a half or quarter of a turn whenever, after considerable periods of time, it sagged too far ou of line, as grate bars always will sooner or later. After some months of use, it was taken out and broken, and found to be a mass of large crystals throughout the whole length subject to heating. To-day after, perhaps, three years' exposure to the air, these crystals remain untarnished; showing either the completeness with which their affinities are satisfied, or the

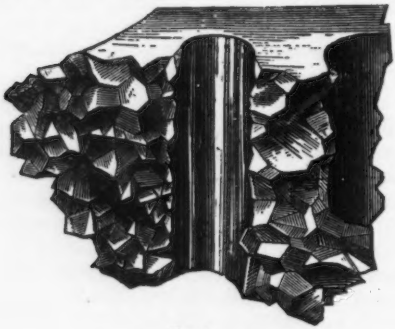


FIG. 1.

FIG. 2.

fact that a crystalline structure gives immunity from oxidation through some other action. At page 581, of Vol. II, of my "Materials of Engineering," is a remarkable illustration of the same sort—a hammer-head of steel, after exposure by accident to prolonged but moderate heat, became similarly a mass of crystals. The cut above is made from a photograph, and is a faithful reproduction. Fig. 1 shows hammer-head magnified; Fig. 2, hammer-head natural size. The effect of heating and forging, as presumed, are very strikingly illustrated by other engravings in that article. The repeated blows of a hammer undoubtedly facilitate this formation of crystalline structure. It seems more than possible that the action of the forging press may prove very much less objectionable on this score; though, after all, at present, it may be perhaps as well that anything that cannot be safely hammered should be rejected.

CHARLES MACDONALD, Vice-President, Am. Soc. C. E. (by letter).—The paper recalls the familiar subject of the relative importance of chemical and physical tests, of which so much has been said and written that it is scarcely necessary to indulge in extended discussion.

Even assuming that the hypothesis as to the chemical constituents of

the bar in question is true, what inference may be drawn from it? Shall the engineer fix limits of carbon, phosphorus, manganese, etc., for the manufacturer, and thereby relieve him of responsibility for the physical properties required, or shall he simply indicate what the quality of the finished product must be, and leave to the manufacturer to determine the best methods of accomplishing the result?

There can be but one opinion as to this. The engineer has to deal with results and not methods of manufacture, and the only satisfactory method of verifying results is by physical tests.

The writer observes that the object of full-sized eye-bar tests is to check the quality of the workmanship, the condition, quality, and grade of the metal.

In so far as the workmanship refers to forging the eyes, this is correct; but the condition, quality, and grade of the metal should certainly have been determined before any expense was incurred in the bridge shop.

This would seem to be a self-evident fact, and yet, judging from some of the best bridge specifications, it is not properly appreciated.

In this connection reference might be made to an extract from the "Hawkesbury" specifications, from which it will be seen that, while the chemical question is ignored, full provision has been made for determining the quality of the material beyond peradventure.

These specifications represent the best practice in England at the time they were drawn up (1886). For brevity and simplicity they are worthy of careful study:

HAWKESBURY SPECIFICATIONS. STEEL WORK. 1886.

"37. The whole of the superstructure will be "Siemens" steel, of a make specially approved by the engineer.

Strips cut lengthwise or crosswise to have an ultimate tensile strength of not less than 30 tons (2 240 lbs.), and not exceeding 33 tons (2 240 lbs.) per square inch of section, with an elongation of at least 20% in a length of 8 ins. The angle and bar steel to stand such forge tests, both hot and cold, as may be sufficient, in the opinion of the engineer, to prove soundness of material and fitness for the service. Strips cut lengthwise or crosswise, 1½ ins. wide, heated uniformly to a low cherry red, and cooled in water of 82° Fahr., must stand bending double in a press to a curve of which the inner radius is one and a half times the thickness of the steel tested. The ductility of every plate, angle, etc., is to be ascertained by the application of one or both of these tests to the shearings, or by bending them cold by the hammer. All steel to be free from lamination and injurious surface defects. One plate or angle, etc., to be taken for testing from every invoice, provided the number of plates, angles, etc., does not exceed fifty. If above that number, one for every additional fifty or portion of fifty.

Steel may be received or rejected without a trial of every thickness on the invoice. In cases where plates or bars have to be heated, the greatest care must be taken to prevent any work being done upon the material after it has fallen to the dangerous limit of temperature

known as "blue heat," say from 600 to 400° Fahr. Should this limit be reached during working, the plates or bars must be reheated.

38. All sheared edges of plates and bars will have at least $\frac{1}{16}$ in. of metal removed by planing, and all holes will be drilled; but a hole $\frac{1}{8}$ in. smaller than the finished hole may be previously punched, if desired.

42. Tests of full-sized bars will be made as follows: Out of every lot of 103 bars, three shall be selected by the inspector; if two out of the three break in the body of bar with a stress not less than 28 tons per square inch, nor a less elongation than 10% in the length of the bar, then the lot of 100 will be accepted, if otherwise satisfactory in quality. If but one out of three so breaks, the testing shall be continued at the expense of the manufacturers until the required proportion of $\frac{2}{3}$ be reached. But if five bars break consecutively in the head without developing the above-specified qualities, then the whole lot of 100 shall be rejected."

It may be added that in the present state of the art it is the exception to find steel eye-bars breaking in the head. The requirements in this regard could, therefore, be materially advanced.

L. L. Buck, M. Am. Soc. C. E.—In all the tests of steel and iron which I have made, I have felt that I was unqualified to judge of the effect of chemical constituents because of not being a metallurgist, but when I had specimens rolled to the various thicknesses and shapes to be used in my work, and having cut test specimens from them, had tested them for elastic limit, stretch, reduction, bending, upsetting and forging, and texture of ruptured section, and have assured myself that only such blows as met all the requirements are used in the work, I have found the result generally satisfactory, and assumed that the chemical constitution must be good. I do not mean to say that I would discourage chemical analysis, but that I should be obliged to leave that to some other person. It appears to me that the proper party to ascertain the best chemical combination is the manufacturer. At any rate, he must attend to it, or he will fail to give satisfactory materials.

I have said in other discussions that with all the tests of specimens to ascertain the physical properties, of steel especially, the engineer should insist upon the inspection continuing through the work in the shop, watching all operations of punching, shearing, boring and planing. Also the forging, upsetting, etc., while the metal is hot, and the testing of occasional full-sized members. His specifications should require the rejection of such pieces as do not behave satisfactorily in undergoing such treatment. This calls for close attention on the part of the inspector, but it will pay for the trouble. It will also cause the manufacturer to look carefully after his chemical combinations.

J. M. Knap, M. Am. Soc. C. E.—One is not at all astonished at the results of these analyses. With the large amount of phosphorus and the large amount of sulphur it seems to me a very natural thing that this should give way, especially if it was treated to a high heat; I think high heat would cause the cracking of the eye-bar.

To go back about 25 years, I might state, to show that the physical properties of iron or steel are more important sometimes than the chemical, an incident that came under my observation. In the case of the St. Louis Water Works, in which I was engaged in building the engines, the specifications called for a certain tensile strength of the cast-iron walking beams. The tensile strength required was, I think, 30 000 lbs. They had heard that the Rodman guns required some such test, and thought for these large beams, 30 ft. long and weighing about 30 tons, if they were of a tenacity of 30 000 lbs., it would be a good thing; it would have been under some circumstances. We tried to change this condition, but the Commission insisted upon it, requiring us to make the beams of iron that would stand this test. We were obliged to make the iron of very high density in order to secure the tensile strength required, but always found there was a great deal of shrinkage under such circumstances, and informed the engineers, and the commissioners especially, that there would be danger in making these beams, but were obliged to do it. We cast one of the beams, and in course of time uncovered it carefully, cooled it slowly, and finally took it to the planing machine to plane off the hubs, when one of the men, who was a very strong, muscular fellow, in chipping off the sinking head, hit it such a hard blow that this walking beam went in two pieces, flew apart and separated about three inches. The force that was required to break that beam, calling for 30 000 lbs. to the square inch, was a good many hundred tons. The thing was tried again and a second beam cast. That gave us less trouble because it broke in two in the middle as we were lifting it out. The web cooled first (causing the periphery or rim to burst with the great shrinkage strain) just as a large fly wheel would break if cooled too rapidly. We tested specimens out of these two beams, and the commissioners were satisfied. We afterwards made the beams according to our own ideas, out of softer iron with a tensile strength of 22 000 lbs., and they are working to-day.

I relate these facts simply to show that practical experience should go hand in hand with theory. The specifications requiring the walking beams to be made of cast iron, with tensile strength of about 30 000 lbs. per square inch, would have been all right for a heavy housing shaft or crank, or for heavy ordnance (of that day) when the proper cooling and annealing processes were provided, So it is, in my opinion, with steel and wrought iron for structural work. Certain percentages of phosphorus and sulphur which may give entirely satisfactory results in some forms may cause failures in others.

The contractor or manufacturer, especially if he assumes the responsibility, should be allowed to use his experience and his judgment in such cases.

Wm. De H. Washington, Assoc. M. Am. Soc. C. E.—It seems to

me there is a medium to be struck between the two sides of this argument; that is, whether we shall have too much chemistry in our specifications or none at all.

What one of the gentlemen just said, may well be taken into consideration. He tells us of a large casting in which the iron was of the requisite tensile strength, but so brittle that it broke of its own weight in one case, and by a slight chiseling with a small hammer before it was put in place in another. Had this been put into position a serious accident is likely to have been the result, as it surely would not have withstood the slightest shock. While this was cast iron and the material we are treating of is steel, nevertheless, it seems to me, the same principle holds good, that some requirement from a chemical standpoint should be made in addition to a tensile strength requirement, to guard against the effect of the natural vibration in a bridge, which must surely have a serious effect, and in the course of a few years on hard or brittle iron or steel, be very likely to render it unsafe, by crystallization, to life and traffic, which it is the engineer's business to guard. It therefore seems to me that some reasonable requirement should be made in regard to the chemistry of all iron and steel for use in bridge work in drawing our specifications for such structures.

G. H. Blakeley, Jun. Am. Soc. C. E.—I think, if the analysis represents anything, that the character of the material was very inferior. I know in my own practice, even where there is no specified limit in chemical composition, I do not permit any steel to be used that is higher in phosphorus than .08. We have proved that by experience, and the law is not to use any steel that is greater than .08 phosphorus. I think that, so far as the specification by engineers of the chemical composition of steel is concerned, they are apt to carry it too far. If they would limit themselves simply to the specification that for acid open-hearth steel phosphorus should not exceed .08 or .07, and for basic .04, I think that would accomplish everything that could be desired. In the furnace I believe that the chemical action of the sulphur and phosphorus go on about jointly, and that if the phosphorus is reduced to .08 the sulphur will be about .05 or .06. We seldom pay any attention to the sulphur, because it always keeps below .06. I believe that every railroad company building bridges should specify a steel that does not exceed .08, and I believe it is their prerogative to use their best diligence to see that the specifications are executed by competent, continuous inspection of every heat of steel.

I have had some experience with large eye-bars in which the chemical composition was entirely satisfactory, and yet they failed just about as badly as these did, but I believe that was due to the size and lack of proper power in the rolling machinery to compress the section in its manufacture. But I believe, if .08 is set as the maximum limit

for phosphorus, that you will seldom have the unfortunate result that was produced at the testing of this bar.

Lest I be misapprehended, I wish to say that I do not mean that the engineer should specify a complete chemical analysis. If he specifies the phosphorus and sulphur, for instance, he must not touch the carbon and manganese, but must leave that to the manufacturer; if he contents himself with the phosphorus, I think he covers the case.

GEORGE S. MORISON, M. Am. Soc. C. E. (by letter).—It seems to me that on many accounts this test is less instructive than is claimed. The bar broke in consequence of visible flaws, as was to be expected. After being strained beyond the elastic limit the fractured head was cut off and a new head forged on the stretched bar. The bar was then reannealed, retested, and again broke in the head, showing flaws. A head forged on a bar which has already been strained beyond the elastic limit cannot be considered a fair test, as before forging such head the metal has been put into an abnormal condition. I have had many bars reheaded and retested, but have found these tests more instructive as to forging than anything else. I have found they could not be depended on as showing the quality of the material. In this case, although the bar broke in the head, it did not break at the most prominent seam. Had it been entirely sound the chances are that it would have broken in the head.

When bars are defective requests are always made by the manufacturer to have them taken for test bars, with the understanding that the test bar is acceptable if it meets the specifications, and that if it fails through some of the visible defects, it shall be thrown out and another bar substituted for it. This is really very objectionable, as it establishes a practice of using bars for tests which do not represent the actual material used. To avoid this I have inserted the following clause in my recent specifications:

"No bars known to be defective in any way shall be taken for test bars, but the bars shall be selected as fair average specimens of the good bars which would be accepted for the work."

Bars with visible surface defects are unfortunate accidents which none of us want; in some cases they are good, but if of steel and the defects are in the head, they are almost invariably bad. One trouble that I have found in steel manufacture is the attempt of makers to make something which, while actually reaching the literal requirements of the specifications, is a cheaper material than the specifications intended to provide for. To prevent this, I always put into my specifications requirements about phosphorus and some other matters, solely to avoid an attempt at making a material which has large chances of failure. The same thing may be said of bars known to be defective. These tests may be instructive, though less so than is commonly supposed, but they amount usually to making tests which have to be duplicated, and to losing time.

WILLIAM METCALF, Past Prest. Am. Soc. C. E. (by letter).—It is difficult to discuss this paper from a steel-maker's standpoint, because of lack of information on just the points that an engineer cannot give.

The analysis is bad enough, but if the steel was not too red-short to work sound, the sulphur would not cause cold-shortness if general experience goes for anything.

The quantity of phosphorus and of manganese would account for any cold-shortness there might be in a steel of higher carbon.

There can be no question that segregation does occur, but I do not think it is necessarily at the upper end of an ingot. I believe it will be most pronounced in that part of the ingot that sets last, and that may be the middle part, or even lower down in some cases.

The absence of any knowledge of the appearance of the fracture makes it impossible to say whether the head was forged properly or not; there may have been a very uneven heat in the steel which would have been sufficient to cause the break.

Again, we do not know how the steel was made; if it was blown clear down, or melted clear down, and then recarbonized back to .21 carbon by the use of Fe Mn, it may have been left rotten and red-short by an excess of oxygen, or nitrogen, or both, which the manganese did not remove.

I believe that few engineers, and not all steel-makers, realize the irreparable harm that is often caused in this way; the steel is full of gases, the ingots are spongy, and the steel is often too red-short to be workable.

By melting, not lower than .15 carbon, or .12 at the least—I prefer .15—and then using just enough Fe Mn to remove as much oxygen as is possible and to give .20 to .25 carbon, a vastly better product will be had than if the same material had been melted clear down and then carbonized to the same point.

In open-hearth practice there is a distinct advantage to the manufacturer in stopping the melt at .15 carbon; it takes less time, less fuel, less wear and tear of furnace.

There is less gas, less danger of boiling, sounder ingots, and, consequently, less scrap.

Whether it would be possible for an expert in Bessemer practice to stop the blow at about .15 carbon, I am not competent to say, but if he could do it, he would gain just as much as can be gained in the open hearth.

I think Mr. Cunningham is right in asking for good chemistry and sound steel; unfortunately, chemistry does not tell us about the oxygen and nitrogen which we know to be harmful when in excess.

In regard to cropping the ingots, as Mr. Cunningham suggests, it would be a serious expense to cut away one-third of every ingot, and an entirely unnecessary one if the ingots were made with any skill.

In good practice, 10% is an abundance to allow for pipe, and beyond the pipe there can be no good reason for cutting away the steel.

It is apparent that in the case under discussion the pipe was all cut out, or it would have shown in the fracture, and then there would have been no need of a discussion.

I will give now two ideal analyses and a physical test of the steel:

Silicon	.007	.009
Phosphorus	.005	.010
Sulphur	.024	.027
Manganese	.060	.050
Carbon	.12	.090

Elastic limit, pounds per square inch	30 990
Tensile strength, pounds per square inch	46 800
Elongation in 2 ins., 82 per cent	41
Reduction of area, per cent	75.85
Fracture silky, ½ cup.	

Here you have a comparatively high elastic limit, low ultimate, and a delightfully high ductility.

The steel was made to order to replace iron, and it answered its purpose.

It works beautifully at a high iron welding heat, and it is so red-short at and below medium orange color that it is a nuisance to handle it.

Surely the sulphur is not at fault here.

To make the steel at all it had to be melted down to nearly zero, and then be recarbonized just enough to make it so that it could be worked.

I have the test pieces before me; there is not the slightest evidence of a flaw in the fracture.

I should dislike to have this steel in a boiler or any structure of mine.

In comparison to this I call attention to 24 tests of boiler sheets, with the mean analysis.

TWENTY-FOUR TESTS OF BOILER SHEETS, APRIL, 1884.

	Tensile strength. Pounds per square inch.	Elongation. Per cent. in 1 in.	Reduction of area. Per cent.
Highest	85 680	27.00	50.5
Lowest	64 640	42.00	62.3
Mean of 24	73 142	33.96	48.0

Quenched from bright yellow heat, only one piece endured the cold bending test.

Mean analysis drillings taken from every sheet and mixed thoroughly:

Silicon	.014	Manganese	.204
Phosphorus	.050	Carbon	{.425 / .446} .435
Sulphur	.023		

Boilers, 42 ins. diameter; 26 ft. long; two 16-in. flues; shell, ¼ in. thick.

These boilers have had ten years of steady, hard work, being pushed often to their highest capacity. They are in perfect condition and look as if they would never wear out.

Such steel would probably not do well in a locomotive fire-box.

It is crucible steel, made by a concern having a high reputation for that sort of work; not ours. Although this steel seems high for boiler work, compared to many specifications that are sent out, I feel much safer with them than I would with boilers made of the milder steel with which they are compared.

The ductility seems good in this boiler steel, but it was measured on 1 in., and the testing engineer notes: "These figures are all probably too high, owing to the shape of the test piece required by the Government."

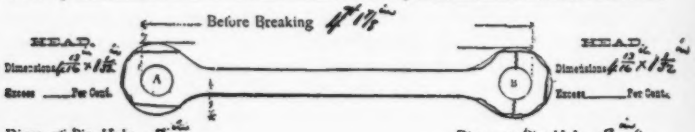
A comparison of the mild steel given above, with some high steel, may be of interest. The three tests, pages 428 and 429 (on Keystone Bridge Co. blanks) in eye-bar form were made at the request of some friends.

TEST No. 1.

FULL SIZED TEST OF 2 × 1 in. Steel EYE BAR.

Rolled by _____ Manuf'd by Crescent Steel Co.

Before Breaking 4 ft 1 7/8 in.



Diam. of Pin Hole 2 in.	Diam. of Pin Hole 2 in.
Elong. of Pin Hole	Elong. of Pin Hole
Nominal Section 2 in × 1 in.	Actual Section 1.91 × 1 in.
Original Area 1.91 sq. in.	Fractured Area
Gauge Reading for Elastic Limit 6.25	= 162,500 Pounds.
Gauge Reading for Ultimate Strength 7.40	= 192,400 Pounds.
Elong. in 42 in. Inches 9/16 in.	Elong. in Feet

ELASTIC LIMIT	85,078	POUNDS PER SQUARE INCH.
ULTIMATE STRENGTH	100,733	POUNDS PER SQUARE INCH.
ELONGATION IN 42 in. INCHES	0.5	PER CENT.
ELONGATION IN FEET		PER CENT.
REDUCTION OF AREA AT FRACTURE		PER CENT.

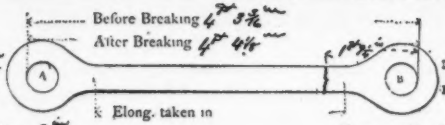
Fracture Broke in head. Close grain

Analysis	REMARKS
Silicon .156	
Phosphorous .008	
Sulphur .015	
manganese .24	
Carbon 1.35	

TEST No. 2.

FULL SIZED TEST OF 2 × 1 Steel EYE BAR.

Rolled by _____ Manuf'd by Crescent Steel Co



Diam of Pin Hole 2in
Elong. of Pin Hole _____
Diam of Pin Hole 2in
Elong. of Pin Hole _____

Nominal Section 2in × 1in — Actual Section 2.01in × 1in
Original Area 2.01in — Fractured Area 1.748
Gauge Reading for Elastic Limit 540 — 120.400 Pounds.
Gauge Reading for Ultimate Strength 910 — 236.600 Pounds.
Elong in 6 Inches 1⅛in — Elong in 2½ Feet 1⅞in

ELASTIC LIMIT 69.850 POUNDS PER SQUARE INCH.
ULTIMATE STRENGTH 117.710 POUNDS PER SQUARE INCH.
ELONGATION IN 6 INCHES 7.24 PER CENT.
ELONGATION IN 2½ FEET 2.85 PER CENT.
REDUCTION OF AREA AT FRACTURE — 13.03 PER CENT.

Fracture Fine, close grain

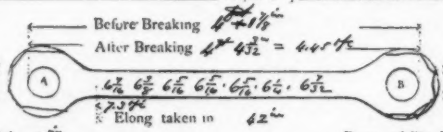
REMARKS

Carbon, Abt. 1.40
Other elements practically the same as No 1

TEST No. 3.

FULL SIZED TEST OF 2in × 1in Steel EYE BAR,

Rolled by _____ Manuf'd by Crescent Steel Co



Diam of Pin Hole 2in
Elong. of Pin Hole 1/16in
Diam. of Pin Hole 2in
Elong. of Pin Hole 1/16in

Nominal Section 2in × 1in — Actual Section 2in × 1in
Original Area 2.04in — Fractured Area _____
Gauge Reading for Elastic Limit 500 — 130.000 Pounds.
Gauge Reading for Ultimate Strength 960 — 249.600 Pounds.
Elong in 42 Inches 2in — Elong in _____ Feet _____

ELASTIC LIMIT 65.000 POUNDS PER SQUARE INCH.
ULTIMATE STRENGTH 124.800 POUNDS PER SQUARE INCH.
ELONGATION IN 42in INCHES 4.75 PER CENT.
ELONGATION IN _____ FEET _____ PER CENT.
REDUCTION OF AREA AT FRACTURE — _____ PER CENT.

Fracture Broken in neck, slight flaw Close fine grain

REMARKS:

Analysis, practically the same as No 1 except carbon

Carbon .96

It will be noted that the .96 carbon gives the highest ultimate. This is in accordance with all of our experience, that steel of carbon .96 to 1.05 is the strongest in all tests except compression.

The absence of ductility in these tests indicates that they are above the safe limit for structural work.

A standard specification for crank pin and side rod steel is :

"Test piece should have a tensile strength of 85 000 lbs. per square inch, and an elongation of 15%.

"Steel will not be accepted if tensile strength is less than 80 000 lbs., nor if the elongation is below 12%."

Carbon.	Pounds per square inch. Elongation limit.	Tensile strength.	Elongation.		Reduction of Area. Per cent.
			In 2 ins.	Per cent.	
.50	63 560	84 220	.50	25	29.91
.60	71 500	108 800	.29	14.5	13.55
.70	69 980	117 400	.23	11.5	8.59
.60	.....	92 650	.28	14	17.32
.60	Slight flaw in fracture.	73 220	.44	22	23.91
.60	.....	81 160	.40	20	26.58
.60	.....	96 800	.30	15	16.26
.60	.....	97 170	.36	18	18.25
.60	.....	94 520	.36	18	16.96
.60	67 980	81 460	.36	18	26.24
.60	78 760	81 500	.26	13	12.56
.60	75 300	111 700	.42	21	27.27
.60	72 390	93 060	.40	20	27.22
.60	75 300	88 460	.40	20	21.88
.60	77 000	93 920	.40	20	20.85
.60	81 940	92 020	.26	13	
.60	75 650	86 760	.26	13	
.60	.....	100 000	.30	15	
.60	.....	104 100	.25	12.5	
.60	.....	93 620	.38	19	
.60	51 300	78 170	.36	18	25.09
.60	47 140	77 440	.42	21	29.39
.60	.....	93 430	.34	17	23.84
.60	.....	94 920	.35	17.5	27.57

We made three samples, .50, .60 and .70 carbon, respectively ; the tests are the first three in the table.

We adopted the .60 carbon grade, and gave the result of 21 tests, representing several hundred forgings. When the forgings were first made we found that the tests were a little high ; to correct this we annealed all of the forgings slightly, and the difference in the amount of the annealing accounts for the wide variations in the tests.

These forgings have given general satisfaction, and when the strains and shocks they are subjected to are considered, it would seem as if structural steel of the same quality might be safe as high as .50 carbon.

The mean analysis of these forgings is :

Silicon.....	.025	Highest.....	.046	Lowest.....	.009
Phosphorus	.016	"	.018	"	.009
Sulphur.....	.028	"	.05	"	.021
Manganese.....	.325	"	.45	"	.26
Carbon	.60	"	.65	"	.56

To test the effect of cold drawing, tempering, annealing and hardening and tempering, we took a piece of ½-in. round cold drawn wire and cut it into four pieces.

Cold Drawn Bar.	Elastic Limit. Pounds per square inch.	Tensile strength. Pounds per square inch	Elongation.		Reduction of Area. Per cent.
			In 3 ins.	Per cent.	
Cold drawn broke in grip.....	92 420	141 500	.06	2.00	2.42
Same bar drawn black.....	114 700	138 400	.18	6.00	12.45
Same bar annealed.....	68 110	98 410	.30	10.00	11.69
Same bar hardened and then drawn black.....	152 800	248 700	.25	8.33	19.7

Analysis.

.15 carbon.

Silicon.....	< .02	Phosphorus.....	< .02
Sulphur.....	trace	Manganese.....	< .30

The upper test is of the piece without treatment.

The next piece was heated until it passed through all of the temper colors to black, and then cooled slowly.

The third piece was annealed carefully.

The fourth piece was hardened carefully and then drawn through all of the color tempers to black.

The analysis is given below.

The figures speak for themselves.

G. Leverich, M. Am. Soc. C. E. (by letter).—The futility of an engineer in his specifications, prescribing conditions not of necessity correlated, is apparent ; thereby he, perhaps unconsciously, exposes a lack of the knowledge upon which his requirements always should be based. As well try to solve an equation of two variables ; impossible if they are independent, and to be done only when their relation to each other is known, and one drops out as unnecessary.

Usually he should not determine the methods whereby his materials are to be produced ; since he is unlikely to be as well informed concerning them as is the steel or iron maker, whose skill and judgment are at his command ; and if he were, the latter confined to a particular formula or process, is thereby deprived of that liberty of choice

and action which will best permit the exercise of his riper and more definite experience and secure a better product. Instead, the engineer should determine results ; by physical tests, he may establish beyond question or doubt, either on his or the contractor's behalf, the precise quality of the material supplied, irrespective of the kind and proportions of its constituents, and which the practiced chemist—much more the engineer, who confessedly is not such an expert—may err in detecting.

Generally, the engineer should deal with that which he knows, and leave the other pertinent matters to those who know better than he ; thereby he will tend to secure confidence in his judgment and prompt acceptance of his decisions. The old rule is still in force : "To do as one would be done by." An engineer has no right, neither is it politic for him, to insert in his specifications, conditions to govern the contractor which, if their positions were reversed, he would be reluctant or unable to observe.

As pertinent to the consideration of material differing widely in physical characteristics, and subject to similar stresses, the following is submitted, deduced from a record *more full than heretofore published, of tests made at the Watertown Arsenal twelve years ago of nine full-sized eye-bars for the lower chords of the Bismarck Bridge, when steel as a material for such members was in less favor than now.

These bars were 25 ft. long; they varied in width from 6.36 to 6.65 ins., in thickness from 0.98 to 1.62 ins., in transverse section from 6.49 to 10.77 sq. ins., and in carbon contained from 0.09 to 0.559%; per square inch of section, their elastic limits varied from 32 540 to 43 310 lbs., and their ultimate strengths from 46 600 to 74 890 lbs.; their extensions, per 260 ins. in length, varied from 0.119 to 0.1337 ins., under a stress of 15 000 lbs. per square inch; from 0.2445 to 0.2797 ins. under a stress of 30 000 lbs. per square inch, and from 4.37 to 27.25 ins. under their several breaking strains.

Diverse as these bars were in section and quality, as thus shown, still having reference to their extensibility as noted, had they been placed side by side to form a single lower-chord member and the same subjected to an aggregate stress of 15 000 lbs. per square inch, the stress on each would have varied from but 590 lbs. below to 960 lbs. above it; and had the aggregate stress been 30 000 lbs. per square inch, this variation would have been but from 1 150 lbs. below to 2 660 lbs. above it.

* "The Bismarck Bridge," a Report by George S. Morison, Engineer, 1884. See Appendix K and L.

From Record of Tests to Determine the Tensile Strength of Nine Full-Sized Steel Eye-Bars, for the Bismarck Bridge, Northern Pacific Railway, made at Watertown Arsenal, January 28th to February 4th, 1882, for George S. Morison, Engineer in Charge.

The bars were of Open-Hearth Steel and the Eyes were Made by the Kloman Process. The Gauge Length to Determine Elongation was 260 ins. The Pin Holes were 5 ins., and the Pins used were 4.98 ins. Diameter.

Dimensions—Inches.			Percent. of carbon.	Strength per Square Inch—Pounds.		Extensions in 260 Ins.			Stress on each bar, if the whole formed one lower chord member and were subject to an aggregate stress per square inch of—	
Width and thickness.	Section.	Length, center, to center of pin holes.		Elastic limit.	Ultimate.	15 000 lbs. strain.	30 000 lbs. strain.	Under breaking strain.	15 000 lbs.	30 000 lbs.
6.62 × 0.98	6.49	300.06	.09	33 900	59 580	.1321	.273	36.23	14 730	29 370
6.55 × 1.13	7.4	300.4	.1285	33 510	59 730	.1292	.272	34.78	15 060	29 470
6.54 × 1.28	8.37	299.8	.178	36 798	66 070	.119	.2455	28.13	15 960	32 660
6.6 × 1.02	6.73	300.08	.18	32 540	60 540	.1331	.2797	26.05	14 610	28 660
6.6 × 1.27	8.38	299.9	.183	32 900	63 240	.132	.28	37.25	14 740	28 630
6.56 × 1.05	6.89	300.75	.19	34 250	46 600	.135	.2779	6.17	14 410	28 850
6.65 × 1.62	10.77	300.29	.203	36 398	59 545	.1267	.251	14.1	15 530	31 940
6.5 × 1.21	7.87	299.9	.222	37 330	62 640	.1279	.2655	21.7	15 210	30 200
6.36 × 1.14	7.25	300.4	.559	43 310	74 890	.1337	.2778	4.37	14 550	28 860

A. C. CUNNINGHAM, Assoc. M. Am. Soc. C. E.—Realizing the incompleteness of the investigation in my eye-bar test, it was with some hesitation that I presented the case to the Society. Believing, however, that a few facts are better than none at all, I decided to submit what little data I was able to obtain through my personal exertions.

The thorough and comprehensive discussions of this case seem to have warranted my venture. What we are after, as individuals, and as a Society, is the truth, and these discussions have brought out a great deal of it.

This eye-bar was of steel, made by a reputable and experienced manufacturer, and was headed and tested by another equally experienced and reputable. Are either or both of these parties to be blamed for this bar? In my opinion they are not. The fault lies in the condition of things as they exist at the present time, and may be laid to incomplete and insufficient specifications, lack of information about steel on the part of engineers, and their difficulty in obtaining the same, and careless, incompetent or underpaid inspection. The conditions are not a little helped by the general desire to obtain a first-class article for one-half its value.

Before proceeding farther, I will ask your indulgence while we go over the generally accepted laws of segregation. In the first place, segregation means "separating out."

When steel cools from a melted condition, some of the mixed or combined elements have a tendency to separate from the hardening metal and remain in the more fluid portions, in the same manner that freezing water has a tendency to free itself from foreign substances. As the steel continues to harden, this separation goes on until the portion which hardens last will contain the greatest amount of the separating elements.

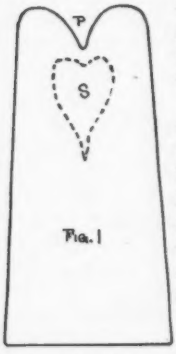


FIG. 1

Among the elements which have a tendency to separate from the hardening steel are carbon, phosphorus and sulphur, while manganese does not have much of a tendency. Thus, three of the most important elements of steel, two of which are the most dangerous, are certain to be unequally distributed throughout an ingot. This variation is not infrequently double between the greatest and least segregated parts.

Let us now consider a normal ingot (Fig. 1). This ingot shows a pipe at *P*, and we have reason to believe that the greatest segregation exists in a pear-shaped space as indicated by the dotted line about *S*. It is readily seen from this figure how an ingot can be sufficiently cropped to remove all piping and still retain the maximum segregation.

I note in the discussions that the opinions are divided as to the desirability of excessive cropping, to remove segregation. The matter can be arranged otherwise, if deemed advisable.

The fast-extending basic process can be called upon to dephosphorize and desulphurize to such an extent that the maximum segregation which can take place shall be harmless. This will lessen the danger from segregating carbon, and, in the meantime, we can search for a new hardener to replace carbon, which does not segregate.

Where acid steels are more desirable, the basic process again will serve to furnish a melting stock for them, so that the final acid steel may be as free from dangerous segregation as the basic.

Attention and study of the size and form of ingots, their rate and manner of cooling, and the casting temperature of the steel, will all lead to the prevention of excessive segregation. I am not aware that these things have received any general amount of attention yet, nor do I think they will until we have insisted upon the excessive cropping, or competition forces the matter.

There can be no valid objection to an engineer specifying limitations to the dangerous elements of steel. Neither should there be any objection to his specifying limitations to all the known elements of steel, provided a reasonable variation is allowed.

Before the introduction of manganese, steel made but little advance, and while we know that this element is a good thing, we have also learned that too much or too little of it may be very bad under certain conditions.

The engineer who has specified chemical limitations on his steel, though better off than the one who has not, must not feel too assured of his safety.

As specifications are generally written, read and translated, the chemical requirements are determined upon the manufacturer's "heat" est. This is a small sample dipped from the still melted charge and which chills so quickly that little segregation can take place; in addition, average drillings may be taken from this sample. This "heat" analysis fairly represents the average condition of the steel, but it is certain that in every ingot there will be steel containing more and also less of the elements than are shown by this test.

Too much reliance has so far been placed upon tension tests alone. I do not question their value, nor would I have them abandoned, but I think it is nearly time for them to step into the rear rank.

When a certain grade of steel is ordered from the manufacturer, he must take into account the chemistry of his stock and the chemistry of his finished steel, and, with certain allowances for work and rolling, his material, in normal cases, fills the physical requirements. From the known analysis he can calculate the ultimate strength within a few thousand pounds, and for him the tension test becomes simply a check.

The engineer who desires a true and thorough knowledge of his steel must follow it from the raw to the finished material. From time to time we hear of new and simple tests to be applied to steel which are to tell us all about it without other aid. Such things must be regarded with caution, for, although they have their value, a true knowledge of steel can not be derived without its comprehensive consideration.

Mr. Roberts' questions indicate an investigation and inspection which would have given us a very satisfactory insight into this case. Unfortunately it was impossible for me to carry the matter out on these or similar lines.

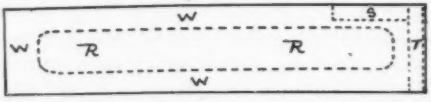
Let me ask—how many engineers have all the information about their eye-bars that is indicated by these questions ?

The general adoption of the chemical limitations suggested by Messrs. Roberts and Blakeley would be a great advance in specifications; they would also practically prohibit the use of Bessemer steel.

I will ask you to note that I do not assert that this analysis is due to segregation; as Mr. Christie suggests, it may have been the general character of the steel. If it was, an examination of the segregated portions would have been an interesting study. The high tensile stress developed in the second testing I think is a proof of the care that was used in forming the second head.

I agree with Mr. Macdonald that the quality of eye-bar material should be definitely settled before the material leaves the mill, and it could be; unfortunately, it is not. Fig. 2 will illustrate this, which

Fig. 2.



we will assume, for an extreme case, to be a 10 x 2-in. bar made on universal rolls. These heavy sections consist of a well-worked shell *W*, surrounding an unworked or raw core *R*, which is coarse and open in its nature, and may be highly segregated.

The specimen tests are taken as at *T* or *S* and represent the well-worked shell and ignore the unworked center; neither will an analysis from them indicate any segregation.

The Hawkesbury specifications would be excellent if we knew all about our steel to start with. As it is, they have the inherent defect of taking no account of the different melts of steel, and it is not an impossible supposition to say that every bar might be from a different melt.

I would call attention to my statement that after the first head broke, there was no elongation or reduction in the body of the bar.

The elastic limit reached was probably at the opposite end of the bar from that in question, and there could have been no serious distortion of the material before the second head was formed. The second head broke in solid material, and not in the flaws, as was expected.

The proper way to restore strained steel is generally considered to consist in annealing it, or heating and reworking it. This bar had both.

Mr. Morison's idea of discarding defective bars, I think, is a mistake. It is all right to require a perfect bar for an official test, but I believe, in addition, that a specification should require the testing of every bar that is defective.

It is from the study of known and visible defects that we arrive at a knowledge of the unknown defects, and will finally learn how to make perfect bars.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

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703.
(Vol. XXXI.—April, 1894.)

THE DEVELOPMENT AND RECENT IMPROVEMENT OF CONCRETE-IRON HIGHWAY BRIDGES.

By FR. VON EMPERGER, C. E., M. Austrian Society of Engineers and Architects.

READ APRIL 4TH, 1894.

WITH DISCUSSION.

The combination of concrete and iron was first used about 16 years ago for building purposes, and it seems to the writer very characteristic that the first use of concrete in bridges, both with and without iron, was begun in the country where the remains of a number of old concrete bridges show that the old Roman "engineers" had something to teach to posterity. It is very likely that this standing example of engineering skill must have impelled the French engineer to works which could be called a "renaissance" in bridge work. They could accomplish more, as the Portland cement of our days is a better material for the purpose than the pozzuolana, which the Romans used in their structures. Some of the latter are still entirely untouched in their exterior finish; as, for instance, the Pont du Gard near Nimes, France, and do not show the interior construction of Roman concrete. The accompanying illustration (Plate LIII), which represents the Aqueduct of Vejus, however, shows that the Roman concrete had larger stones than we use now, but laid without dressing in a bed of mortar and gravel.

PLATE LIII.
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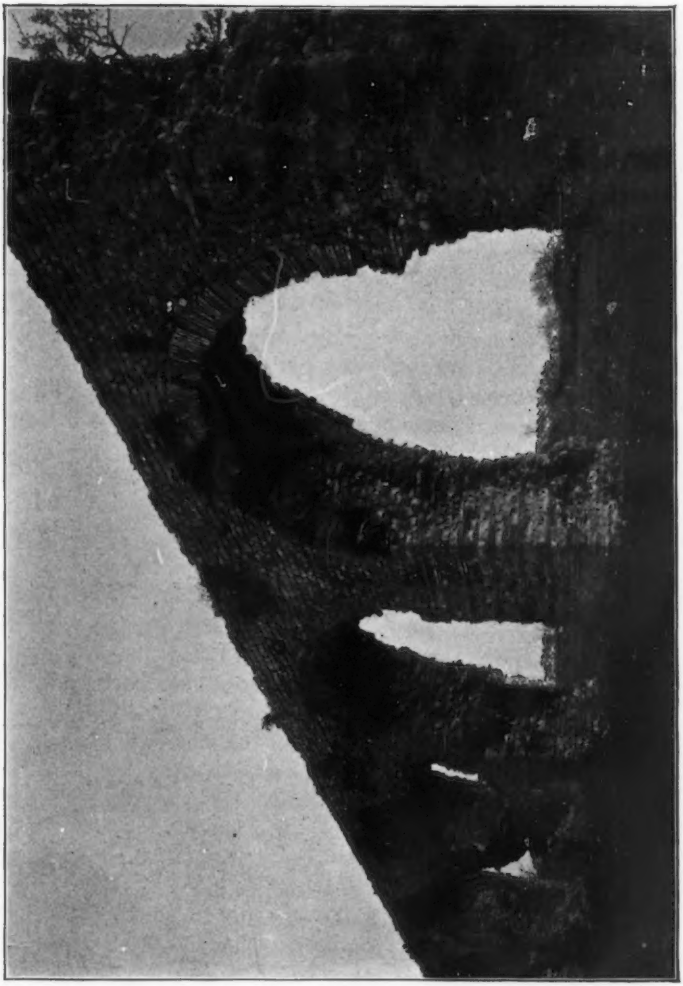

While the use of concrete for bridges soon became general in the central part of Europe, its application was not common in other countries and advanced slowly with the progress of the manufacture of Portland cement.

In this country, for instance, the use of concrete has been almost confined to floors, and the writer would especially mention, Phineas Ball, M. Am. Soc. C. E., of Worcester, Mass. (1884); Mr. Ransome, of San Francisco (1884); and Mr. N. Poulson, of New York (1892), as among Americans who have improved and used concrete with iron in floor construction. The number of concrete bridges is very small, and the writer knows of but three concrete-iron bridges in this country. In view of this fact, he believes that some of the experience had in Europe with this combination will interest American engineers, as well from the historical as from the engineering point of view.

The man to whom the invention of this combination is generally attributed, is Jean Monier. He lived at the beginning of his peculiar career (1876) near Paris, and was in the gardening business. His intention was to build large and strong tree-pots, as they are much used there. Monier used for this purpose concrete with imbedded wire nets, and invented the so-called "Monier System." We see that he was far from being a trained engineer, but had an inventive head, which often is worth more than training. It is idle to state that he was not the discoverer of the process; but he perceived the value of his method, developed it with the help of trained engineers and capitalists, and, what is always the most important, he made a success of it.

We hear first of him when he received a 1st class Diploma of Honor at the exposition in Paris (1878), and, in a comparatively short time, he built nearly 1 000 water and gas tanks throughout France, some of them 70 ft. in diameter and 11 ft. high. The next application of this system was to tubes and sewers; then came vaults and bridges, and finally flat floors.

In France and Germany, where the use of concrete for bridges was common, even before that time, the application of this improvement found its best field, as in those countries it was not necessary to create public confidence in the method, and builders were easily made familiar with this application.

This is, however, the serious difficulty which has to be overcome in other countries.

It is, therefore, not out of place to remember that quite a number of concrete structures, in addition to the Roman water conduits, have stood through centuries; as, for instance, the cross-arched vaulting (78 ft. span) over the bath of Diocletian, in "Hexædra amplissima," and the parabolic main aisle of St. Peter's (87 ft. span and 150 ft. rise), both in Rome.

Speaking of modern structures, the writer may mention the bridge over the Isère, France (span, 85 ft.; rise, 8.5 ft., and 2.5 ft. thickness in crown); the bridges over the Saevale and the Iregua near Torio, Spain (spans, 33 ft.; rise, 7.5 ft., and 3 ft. thickness in crown), and the aqueduct at Vannes, France, built in 1868–69.

The last-mentioned structure has been described in General Gillmore's excellent book, "Coignet Béton." The arches have 75 and 125-ft. span. Later on, the Pont d'Alma and Pont Napoléon were built as railway viaducts near Paris, spans about 140 ft., and one-tenth rise. Another more recent example (1873) is a bridge of 105 ft. span, 13 ft. rise and 20 in. thickness at the apex, near Erlach, Germany. Its designer, Highway Inspector Koch, provided it with three joints, of asphalt plates, so it was turned to a three-hinged arch —an example which has been repeated later several times with lead plates or ball joints. But it seems notable that at this time (1873) any one could have the faith to build such an arch on soft ground. The arch settled, as provided, 5 ins. without injury. It is impossible in the limits of this paper to continue the line of examples of concrete bridges, but the writer may be allowed to mention the bridge built by Dr. Leube, near Ulm, Germany (1885), on account of its bold dimensions. The span is 150 ft. and the thickness in crown is only $6\frac{2}{3}$ ins. Such figures surprise even the engineer, for if these bridges show their proportions plainly, our eyes have to grow accustomed to these delicate forms.

We often hear it praised as great progress that, by the use of Portland cement mortar, stone bridges can be reduced in size 10 per cent. Experience in Germany has proven, however, that with concrete and its monolithic properties a saving of 25% is attainable. A stone arch must be constructed for pressure only; in concrete, however, one-tenth of the pressure (Engesser says one-seventh) may be taken up as tension limit. To illustrate the difference between masonry and concrete-iron construction Plates LIV and LV are given, and show the highway

PLATE LIV.
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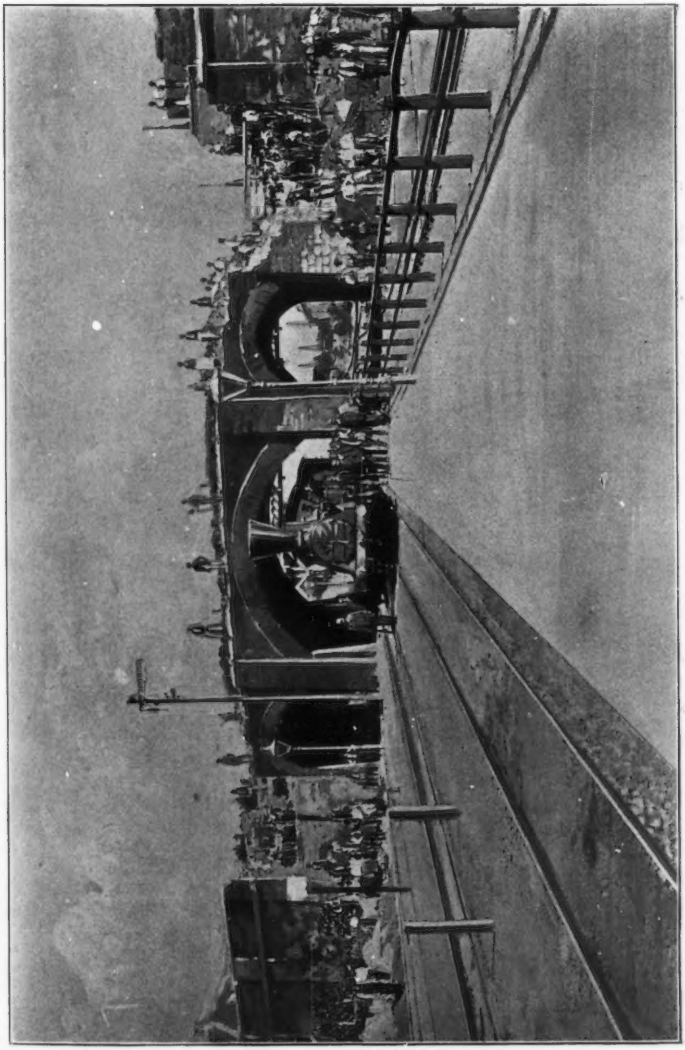

PLATE LV.
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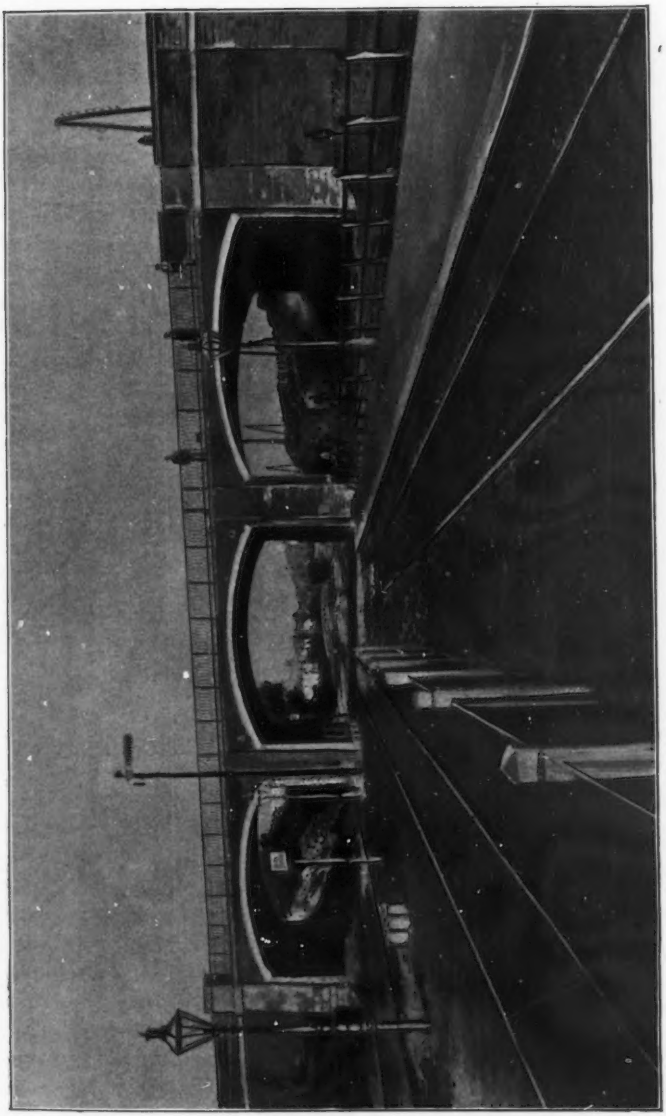


bridge over the railway station at Moedling, near Vienna, as it was originally built, with masonry arches, and as it is now (after 1889), with the Monier system.

The leading idea of the use of iron with concrete is to strengthen the monolithic concrete block against tension, that the unavoidable waste of material involved by a low tension limit and a ten times higher pressure limit may be overcome, and to enable us safely to rely on the tension in the combination. This enables us to build flat as well as thin arches, saving 67% and more, in comparison with stone.

The genuine Monier system had only one wire netting on the intrados, but engineers soon became aware that this was not sufficient. Tension occurs, also, on the intrados at *B*, as well as in the extrados at *A*, Fig. 1.

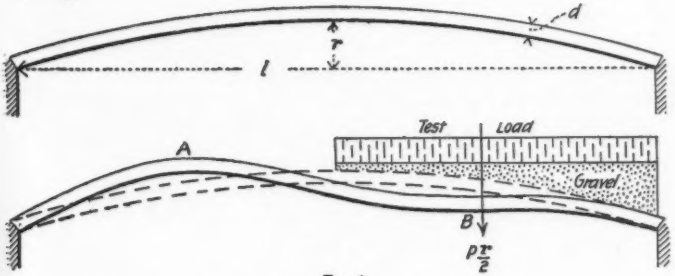


FIG 1.

Two nettings were, therefore, used in larger spans, as shown in Fig. 2, which is a reproduction of tests made by the Building Department in Berlin, Germany, 1886.

From the great number of tests made, to get the real value of this invention, may be noted those of G. A. Wayss & Co., the owner of the so-called "Monier patent," in Germany; also, those of the late Professor Bauschinger, in Munich (1887), and those of the Austrian South Railroad (1889), and others.*

All those tests clearly showed the superiority of this combination over any other arch construction of brick, stone or concrete alone.

As most of the prominent Monier bridges are known through the technical press, the writer only reproduces one (Plate LVI) which has a span of 132 ft., 14½-ft. rise, and is 9⅞ ins. thick at the crown. It was

*See "Die Monierbauweise," by G. A. Wayss, A. Seydel, Berlin, 1891.

designed for a live load of 200 lbs. per square foot, but was built for show purposes only, in a fair (1890) in Bremen, Germany. It may be added that the whole work (concrete approaches and stairs) was finished in six weeks; the main span in 36 hours. The number of Monier

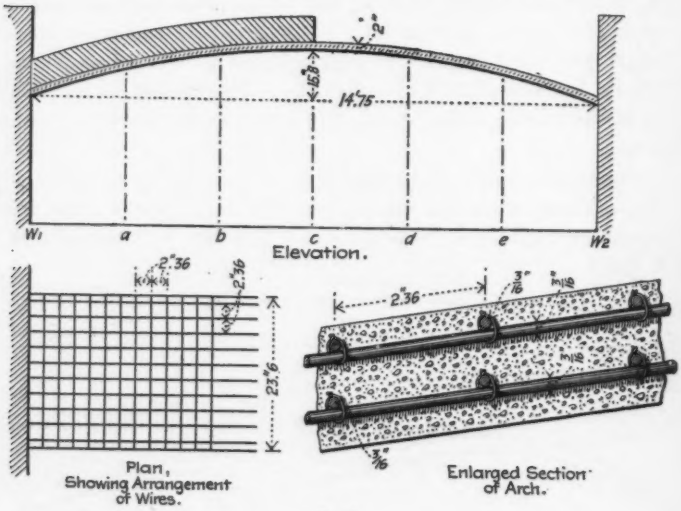
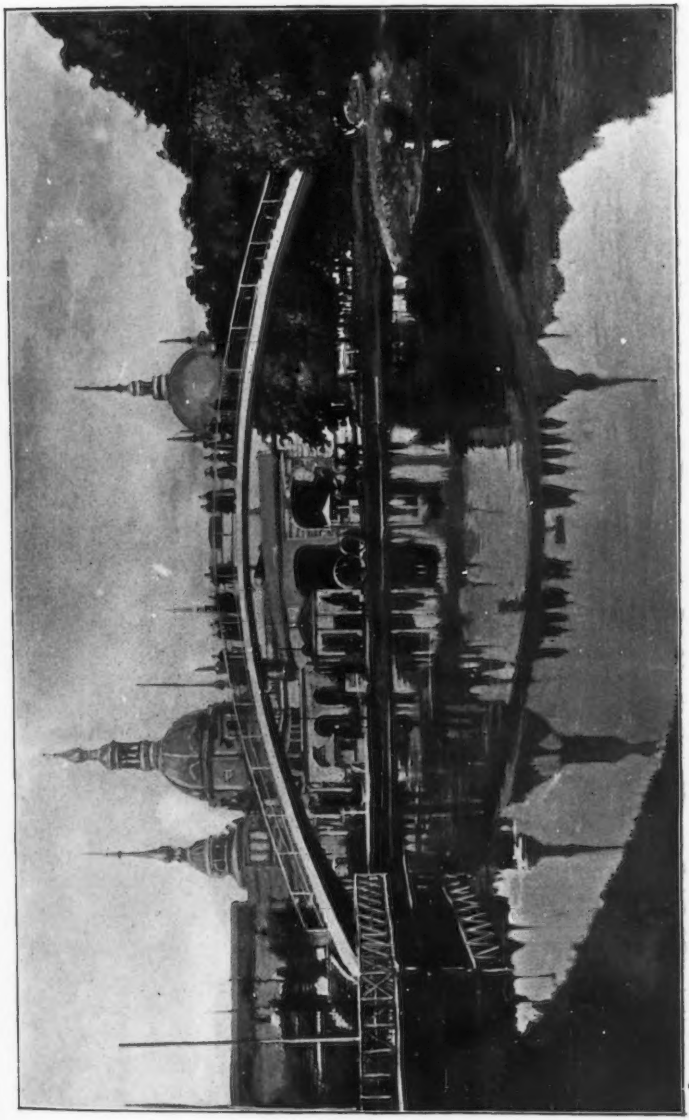


FIG. 2.

bridges in Europe is very large, and the writer's investigation outside of Germany and Austria is not complete. In 1891, some larger Monier bridges were built in Switzerland; one at Wildegg and two at Aarau. All three have 128 ft. span, 11 ft. rise, and are $6\frac{3}{4}$ ins. thick at the crown and 10 ins. at the abutments.

The iron netting used with the Monier system is not stiff itself, and its co-operation with the concrete can be expected only after hardening; but if the work is carefully done, this is of no importance. But what will happen if the work is not well performed? In common building practice it is not extraordinary to find a dishonest contractor who uses an improved kind of dirt instead of concrete, or who hurries the centering away to another job. There is no material which is not liable to failure under such circumstances. Such a case happened just in the beginning of the Monier system in bridge work; the arch fell without warning, and several men were killed. This failure caused



general distrust, and a remarkable set of tests was needed to show that it was not the fault of the system itself.

Nevertheless, this difficulty has been overcome by the other systems now in use, which use rolled shapes instead of wire netting.

Breaking tests have shown that any fault in the concrete must show by distinct cracks, and an overloaded bridge of that kind will go down slowly, as the stiff rods must break first. The only injured person in case of dishonest work will be the contractor. The inventor of this latter method is R. Wünsch, of Buda-Pesth, Hungary (1884). Mr. Wünsch has built with his system six highway bridges, all in Hungary, as follows:

Name.	Span—Feet.	Rise—Feet.
Belovar	41⅓	3
Nyeresfalva	33⅓	3⅓
Munkacs	13⅓	1⅓
Kaschan	27	3⅓
Ladomer	14⅔	1⅓
Neuhausel	55⅔	3¾

The first of these is shown in Plate LVII.

The last one has been fully described in the *Zeitschrift*, of the Austrian Society of Engineers and Architects, and reprinted by the American technical press. The principal used by Mr. R. Wünsch is shown in Fig. 3, which represents a test made with a 16.6-ft. bridge by the Hungarian government, and the results are given on page 444.

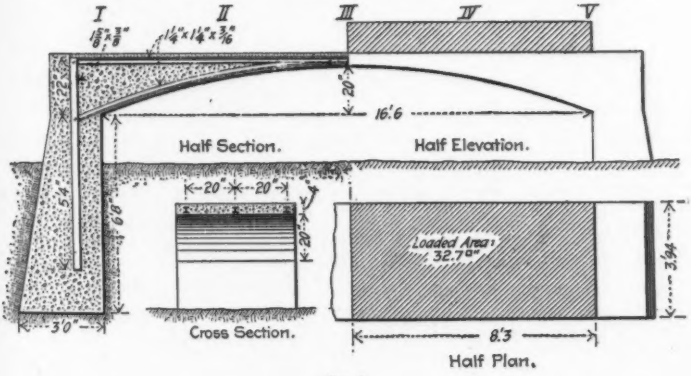


FIG. 3.

Total load.	Load per square foot.	Deflections at Point.		
		II.	III.	IV.
Pounds.	Pounds.	Inches.	Inches.	Inches.
2 844	88	....	....	0.04
5 049	105	....	0.04	0.04
8 356	258	....	0.16	0.16 *
11 673	360	0.04	0.27	0.27 †
16 082	470	....	0.59	0.55 ‡
18 287	565	0.75	2.64	1.97 §
33 101	1 010	....	15.75	....

The structure was designed for a live load of 80 lbs. per square foot.

Two points in every section were closely watched, as well vertically as horizontally. There was no measurable horizontal motion of the abutment, and the inventor claims it as the most important property of this system, that the horizontal thrust is changed to a lift by the anchorage. This would enable us to place the Wünsch arch on top of a high pillar, as well as a flat girder.

The latest improvement in this line is called after the inventor, "System Melan," Fig. 4. It consists of bent I beams, with a concrete body between and no cross-rods at all. It is a very remarkable saving of iron, and, at the same time, an advance in strength. The beams are, of course, the stiffest shape to be had. They are a kind of centering itself. It is not difficult to show that these bent beams would safely carry alone the load intended, and it is hard to understand why this simple construction has not been thought of before. Simplicity in all details is a property which will certainly be appreciated in this country. No strain sheets, no worked-out plans, are needed. All that we must know are four figures—span, rise, section of beam and distance between. The bending is done in the mill at a cost of ½ cent per pound or less, and there are no other connections but splices to prolong the beams. All that the workman has to look after is to keep the beams flush. After these are once in place, the centering can be finished in accordance with the given line. The centering need not be heavy; it has not to support the entire structure, as with other concrete systems,

* Between Sections I, II and IV, V, fine cracks.
† Crack unchanged.
‡ Crack open to 0.08.
§ Cracks increase slowly until the breakdown.

PLATE LVII.
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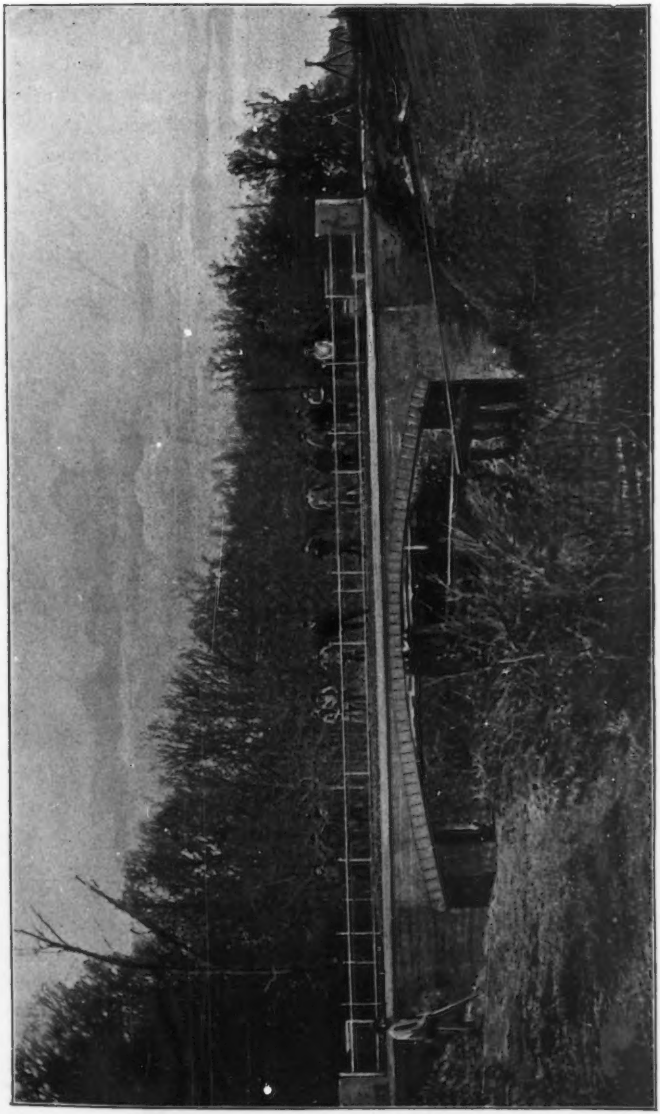


or with stone arches until the keystone is laid. It supports only one ring, and that only partially, until the concrete has set. The work does not need to be hurried through, as in the example given before of the Monier arch (132 ft.), which was finished in 36 hours. The concrete can be laid in rings between the beams. It must be laid in layers of about 6 ins. and well rammed. The inventor of this system, J. Melan, is, in contrast to J. Monier, a well-known engineer

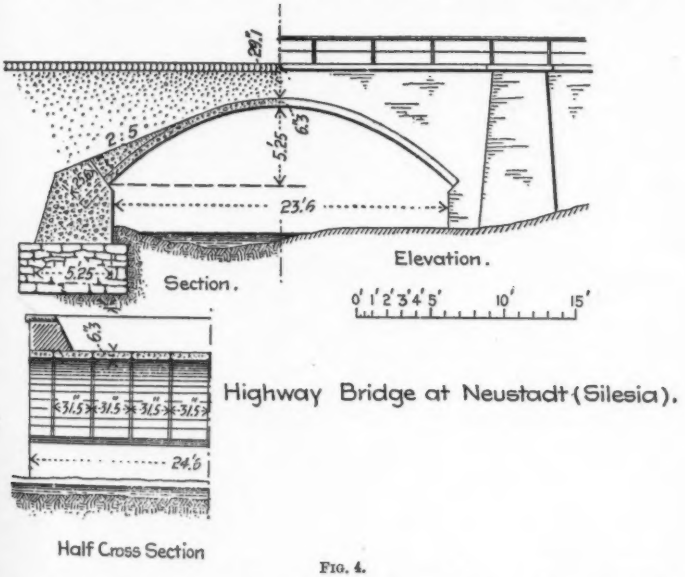


FIG. 4.

and writer. He ranks in German engineering circles as an expert in arch construction,* and served for the Austrian government as an expert on all the different tests made there of late years. He consequently has had ample opportunity to form an opinion, better, at least, than those who have gathered their experience behind a desk. In looking over these three systems, it is a very natural question, which one gives us—with the same amount of steel and concrete, the strongest arch? It must be remembered, that any of them will do, because both materials will do the work as well alone, and the question of

*See "Handbuch des Brückenbaues." Chapter: "Iron Arched and Suspension Bridges." By J. Melan.

application is only one of economy; and that there will be places where cement and labor can be had so cheaply that there is no saving in using iron. On the other hand, there are locations where sand and gravel cannot be had within reasonable distance, and an iron bridge must be advisable under any circumstance. Such a question must presuppose average circumstances.

The Austrian Society of Engineers and Architects undertook a very important step to get at the merits of the existing system of arches. As a measure of the importance of these tests, the fact may be mentioned that they cost 20 000 florins, while the work done by manufacturers without pay had a value of 15 000 florins. The writer's experience in the commercial values of the two countries enables him to say that these tests could be repeated here at an expenditure of $36 000.

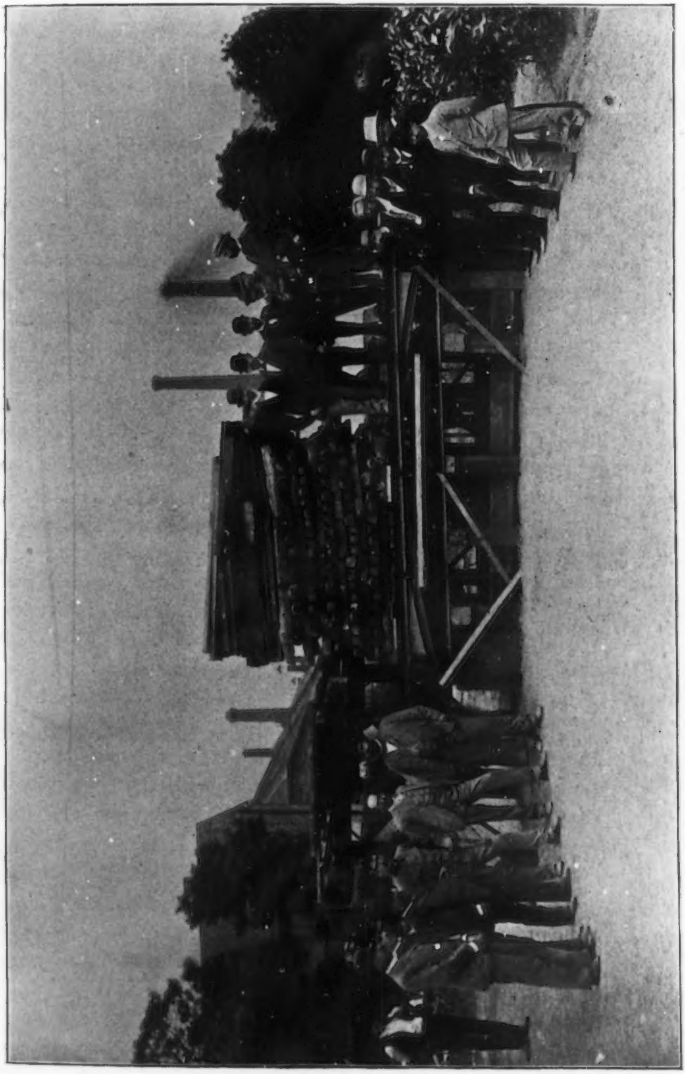
They built arches of all kinds and sizes (from 7 to 77 ft. span), loaded them and broke them down, to get the elastic dislocation and breaking loads. The tests were superintended by a board selected from the principal railroads, builders and colleges, whose duty it was to see that the arches were not built either better or worse than in common practice.

In the following table the four first tests are from this series, while the last one was made by the Hungarian government (1891), as above stated :

	Material.	Span. Feet.	Rise. Inches.	Thickness in crown. Inches.	Steel. Pounds per square foot.	Breaking load. Pounds per square foot.
Fig. 1.....	Brick with cement mortar.....	13½	15¾	6	....	321.5
Fig. 1.....	Concrete.....	13½	15¾	4	....	737.3
Fig. 1.....	Monier.....	13½	15¾	2⅜	4	839.7
Plate LVIII.	Melan.....	13½	11.0	3⅛	1.4	3 360
Fig. 3.....	Wünsch	16.4	20.0	4	1.5	1 010

All arches were loaded as shown on Fig. 1. The Monier arch had one wire net $2\frac{3}{8}$ x $2\frac{3}{8}$-in. meshes out of $\frac{3}{16}$-in. rounds (see Fig. 2), while in the test of the arch of 77-ft. span, referred to above, two wire nets were used.

The Melan arch, $3\frac{1}{8}$-in. I beams, 4.3 lbs. per foot, and 40 ins. apart, and the structure was one year old when tested.



The Wünsch arch had $1\frac{1}{2} \times 1\frac{1}{2} \times \frac{3}{16}$-in. angles with the poorest concrete, and was tested after one month, which explains its lower figures (see also the record of its load, which has been given before).

The test arch, System Melan, is shown in Plates LVIII and LIX. It was designed for a total load of 200 lbs. per square foot, and nobody expected so high a breaking load. Plate LVIII shows 1 500 lbs. per square foot. As no more pig iron was at hand and a margin of seven was reached, it was the intention of the Board to fulfill the programme and to break the arch down.

It was decided to load only one rib in the most dangerous manner by a square of 10 sq. ft., as can be seen in Plate LIX, which shows the broken-down arch.

The first cracks appeared with 3 000 lbs. per square foot, and the arch broke slowly down under 3 360 lbs. per square foot. This is certainly a load which the $3\frac{1}{8}$-in. beam alone never could stand. The safe load of this I beam (flat span as above, $13\frac{1}{3}$ ft.), is 73 lbs. per foot run, and it carried 11 200 lbs. per foot run, or 153 times more; but if we compare it with the bent I beam of the same span we will see that this beam is strong enough to carry the safe load alone. The high degree of safety involves, without doubt, a waste of material, and it is very likely that in the future the stresses used in the calculation may be raised. On the other hand, in the writer's opinion, a coefficient of safety of 5 is not sufficient, and 10, as Trautwine uses with stone, is entirely warranted. The figures used in the arch calculation for tension of concrete vary greatly. This shows mainly, that it depends more upon the theoretic than on the actual stresses, and there is a future for some coming experimenter. Professor Engesser recommends in accordance with tests of concrete arches made by Dyckerhoff & Widman, the use of 213 lbs. per square inch. But this figure leads to paper-like dimensions. Mr. A. Rella recommends 125 lbs. per square inch; while J. Melan, in connection with his calculation given further on, uses 60 lbs. per square inch. The latter figure is certainly too small. The breaking load, so obtained, shows a coefficient of safety of about 15.

The strength of concrete in tension can be taken from 290 to 340 lbs. per square inch. We see from this that Melan uses a theoretically warranted factor of safety of 5. But this proportion only applies to the limit of elasticity, and while we know that a concrete beam breaks under about double the theoretically calculated breaking load, we find

the figure of Professor Engesser explained if we suppose that an arch will break under treble the theoretically calculated load.

On this supposition Professor Engesser has a safety factor 5 times and Professor Melan one 15 times that of real safety, as stated. It would exceed the limits of this paper to say more about this matter; it seems sufficient to add that for the cement mortar commonly used with concrete in bridge work (1 to 3) it is necessary to specify a tensile strength of at least 200 lbs. after 28 days.

All this considered, we have to adhere to the calculation for concrete iron arches, of Professor Melan, as the best for the present until something better shall be at hand.

Calculation of Concrete Iron Arches.

We call—

The dead load in pounds per square foot $= q$;
" live " " " " $= p$;
" span in feet $= l$;
" rise in inches* $= r$;
" thickness of concrete in inches $= d$, this generally equal to height of beams;
" distance between beams in inches $= a$; and further:

	Concrete.		Steel.	
Modulus of elasticity.....	E_1	750 000	E_2	30 000 000
Section.....	A_1	$a\,d$	A_2	
Moment of inertia.....	I_1	$\frac{a\,d^3}{12}$	I_2	
Moment of resistance.....	R_1	$\frac{a\,d^2}{6}$	R_2	
Coefficient of distribution.	α		β	

$$\alpha + \beta = 1.$$

Thesis.—The stresses in both materials, or, what is practically the same, the total loads L, can be assumed distributed, $L = L_1 + L_2$ if it answer the following proportion:

$$\frac{L_1}{L_2} = \frac{E_1 I_1}{E_2 I_2} = \frac{d^3 a}{480 I_2} = \gamma \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

* The rise r is expressed in inches for simplicity only, the results desired being in square inches.

PLATE LIX.
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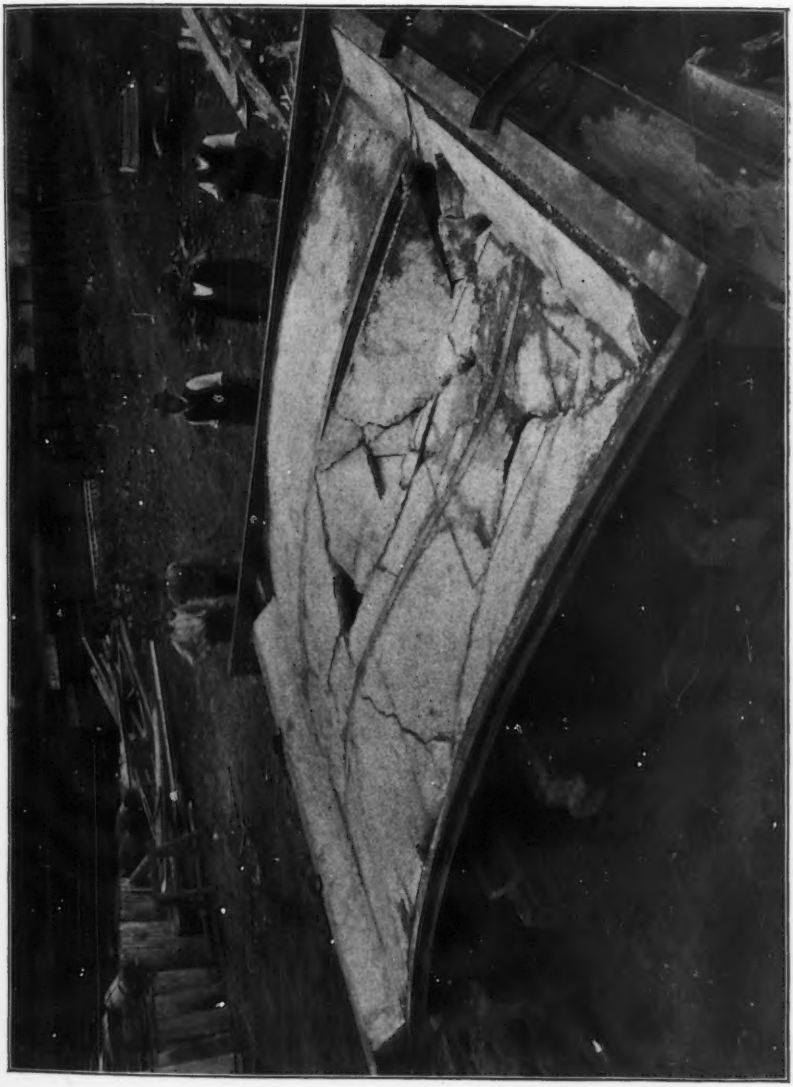


This put in—

$$L = L_1 + L_2 = \alpha L + \beta L = \frac{\gamma}{1+\gamma} L + \frac{1}{1+\gamma} L,$$

or

$$\alpha = \frac{\gamma}{1+\gamma} \text{ and } \beta = \frac{1}{1+\gamma} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

The span and rise being given, we have to assume the size of beam d, and its distance a. The proportion $\frac{a}{d}$ is limited by the necessity that the concrete body has to act as a main and also as a cross-beam. It carries $L_1 = \alpha (q + p)$ as a main girder, and transmits $L_2 = \beta (q + p)$ to the iron beam.

With reference to the elastic theory, in cases where spans in two vertical plans occur, by reducing the allowed stress in concrete to 42 lbs. per square inch, we get—

$$\frac{a}{d} = < \sqrt{\frac{\frac{4}{3} \times 42 \times 144}{\beta (q+p)}}$$

$$= < \sqrt{\frac{8\,064}{\beta (q+p)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

If we find the a of our choice is not too large, we proceed to get the stresses in the main girders. A power H, which is not centrally applied to a section A, produces consequently a moment M, with the following stresses in the outer fibers :

$$S = \frac{H}{A} \pm \frac{M}{R}$$

we only have to use our signs tabulated above and we find the stress per square inch in the concrete :

$$S_1 = \alpha \left(\frac{H}{d} \pm 6 \frac{M}{d^2} \right) \ldots\ldots\ldots\ldots\ldots \quad (4)$$

and in the steel beam—

$$S_2 = \beta \left(\frac{H}{A_2} \pm \frac{M}{R_2} \right) a \ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

It can be seen by a glance, that by proper choice of α and β, which mainly depend on a and I_2, the tension in the concrete or the pressure in the iron can be reduced to its proper limits.

To get the highest tension and pressure, we must consider as previously stated a half-loaded span (Fig. 1).

The horizontal thrust is equal—

$$H = \frac{1}{8}\left(q + \frac{p}{2}\right)\frac{l^2}{r_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

r_1 we call the rectified rise

$$r_1 = r + \frac{15}{16}\,\frac{d^3\,a + 12\;I_2}{(d\,a + A_2)\,r} \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

The moment in a section $\frac{3}{16}\,l$ apart from the center is—

$$-M = \frac{9}{1\,024}\,p\,l^2 + \frac{1}{3}\,H\,(r_1 - r) \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

For a full loaded span—

$$H = \frac{1}{8}\,(q + p)\,\frac{l^2}{r_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Moment in crown—

$$M = \frac{1}{3}\,H\,(r_1 - r) \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

Moment in abutment—

$$-M = \frac{2}{3}\,H\,(r_1 - r) \quad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

The last three equations are of use so far as the spandrels will be considered, while for ordinary use the equations 1 — 8 will be sufficient.

For the abutment itself we have to compute H from equation No. 9 with the weight of it, and to see that the resultant keeps within the inner third; and further, that the total weight has a proper area in the foundation.

Compared with other arch calculations this is extremely simple, and correct enough for practice as a comparison with an exact method, which we get by introduction of moment of inertia and the section in equation No. 1.

The calculation is, of course, not so simple as with a flat beam, especially as we are familiar with mill books. It is the intention of the writer, as soon as time will allow him, to tabulate the results in the same way.

On the other hand, if this calculation is once made, no elaborate plans or strain sheets, and only four dimensions, are needed, viz.: span, rise, beam and distance.

It should not be overlooked that in the foregoing calculation the modulus of elasticity of concrete (such a concrete as we use for bridges) is taken at 750 000. This is about half the figure we have been in the habit of assuming; see, for instance, General Gillmore and others who all give their figures with a great reserve, and some of

them directly doubting their correctness. This figure is the result of the experiments made by Professor Boek in Vienna, Austria, which were recently made in connection with the great undertaking of the Austrian Society of Engineers and Architects mentioned above, and is important as confirming the results of previous tests of beams of concrete alone, as well as in combination with iron.

This investigation was not confined to the arch alone, but extended also to the material used in its construction. The great amount of matter explains why the final report of the committee, although well known in engineering circles in Austria, has not been published yet. The writer is informed that it will appear next month, and it can then be found by those interested.

Two concrete structures which give the best proof of the elastic properties of this material may be mentioned. One is a chimney 160 ft. high, in Ireland, in one piece of concrete, which has stood the heaviest storms, and another example, which is cited by Mr. A. Rella, are wine tanks of a capacity of 80 000 galls. (of concrete only) in Agram, Hungary, which stood the last earthquake without cracking. A further proof is the use of concrete in railroad structures directly below the tie.

There is this excuse for a novice, that, in speaking of concrete, he thinks only of the glazed claypot, but if technical papers speak of concrete as "inelastic, like cast iron and similar materials," this is an unpardonable error. The only difference between concrete and so-called elastic materials is a higher tension limit, and that the elastic limit and the breaking limit do not coincide in the former; they do in the latter. But in no case should the elastic limit in any material be reached, and as long as it is not reached its elastic behavior is the same in all materials.

Concrete and its preparation is to-day so well known that it would be a waste of time to go into the subject. The slightest doubt can be satisfied by reading the chapters relating to this matter in Trautwine's or in other more special treatises. That concrete must be prepared with care to develop fully its properties is not the question, because the same must be said of every building material. But with concrete the work can be superintended by one reliable man, while the actual work can be performed by inexperienced workmen.

The proportions of concrete are varied in accordance to its use as

mortar, as a water-tight shell, as merely a filling (where we sometimes use 1 to 18), and finally where it has to bear stresses.

The materials used for the last purpose are slow-setting cement, sand and gravel or broken stone. The figures 1 to 8 mean the proportion of cement to the aggregates (sand and stone together), but it is of equal importance to know the proportion of the mortar (cement and sand) to the stone, and of the cement to the sand. In both cases the voids, which should be at least filled, to produce a concrete fit for our purposes, must be considered. The proportion will consequently depend upon the size and form of the aggregates on one side and the fineness of grinding and specific weight of cement on the other.

Even in the same bridge we find, if economically built, different proportions used in different parts.

The best concrete in crown and in the outer fibers, 1 to 5; in the haunches, 1 to 7; and in spandrels, 1 to 9. These figures are the highest the writer has heard of, the usual practice being always lower, viz.: crown, 1 to 8; spandrels, 1 to 12.

One to 3 to 6 is recommended by Mr. A. Rella, an expert in this kind of work, wherein 1 to 3 means the mortar and 6 gravel; if broken stone is used, which for that work is the more desirable material, the proportion is changed to 1 to 3 to 5, in which the stone (5) is made up of 1 part of gravel and 4 parts of broken stone. All Monier work has been done with 1 to 3 mortar, no stone being used. Wünsch, on the other hand, sometimes used 1 to 15; but such reduction needs some experience in handling and choice of materials, and cannot be recommended generally. As the strength of concrete mainly depends on the mortar and its proportion to the stone, we specify for a mortar 1 to 3, 28 days old at least, 200 lbs. per square inch, and also the size of stone. It is, of course, quite faulty to consider neat cement tests or tests of only a few days. These structures are built for centuries, and we know that the strength of concrete is always increasing. After about 15 weeks the increase is not of material importance, but after two weeks the strength should be sufficient to carry the dead load of the bridge. On this account we sometimes see the centering removed after two weeks, sometimes after two months; but in no case should a concrete bridge be subjected to severe tests before three months, for it is clear that it will not have developed its full strength before that time.

A very striking example of this was the result of the removal of the centering of the bridge near Neuhausel. The centers of the six spans were taken down at different times. The deflection caused by the dead load was measured very accurately, and is as follows:

Span.	I.	II.	III.	IV.	V.	VI.
Time for setting in days.....	31	32	36	40	41	43
Permanent deflection, inches.....	0.59	0.67	0.55	0.47	0.24	0.08

The deflections show clearly the influence of time. Engineer Schustler recommends for this system six weeks for setting. In the loading tests made for longer periods no further permanent deflections are found.

We may now consider those properties of iron and cement which make both materials especially fit to be used together:

First.—The modulus of elasticity of concrete (after Professor Boeck) is about $\frac{1}{10}$ of that of mild steel. This great difference enables us to have in two neighboring fibers a small stress in the concrete and a 10-times larger one in the steel.

Second.—Professor Bauschinger found the cohesion between iron and concrete, after hardening, to be from 570 to 640 lbs. per square inch. This is more than the tensile strength of the best concrete. It acts like a glue if once hardened.

Third.—Concrete is the best conservator of iron. The writer knows of a case where iron rods were found perfectly rust-free having been imbedded in concrete below water level for 400 years. There are many devices designed on this important property, and for the protection of iron concrete is better than paint.

Fourth.—The thermic expansion of both materials is the same, and change of temperature will not originate secondary stresses.

Bonniceau, a French author, gives the thermic expansion of Portland cement as 0.0000143 for 1° Celsius, while iron has 0.0000145, which is practically the same.

Having now reviewed the properties of the materials used, and their application, as well as the calculation for these arches, there remains to be said something of their usefulness for highway bridges.

The main advantages of this construction are:

First.—That there is no expense for maintenance of the bridge, the iron being entirely covered.

Second.—That there is no expense necessary for maintenance of the road, or, at least, no more than for any other part of it.

Third.—That there are no vibrations and practically no noise, and that it is not affected by a change of live load.

Fourth.—That such bridges are tornado and high-water proof.

Fifth.—That they have a solid appearance, which can be architecturally developed in accordance with surroundings.

Sixth.—That their construction is cheap wherever sand and gravel are at hand.

In short, they possess all the advantages of a masonry bridge, at the cost of an iron one.

But all the first five merits have little value without the last one. In practice, prices rule the application. Can any one deny that the very best examples in bridge-building are masonry constructions? and yet how seldom do we see one.

It is certainly remarkable, therefore, that Mr. Trick, Superintendent of Bridges, Philadelphia, in building his concrete-iron bridge on Pine Road, states, like many other authors, that concrete has been for him a cheaper material than stone. On the other hand, we have the fact that the bridge near Neuhausel has been awarded to the concrete bidder, notwithstanding that the Government asked only for bids for a wooden bridge. The estimate for the wooden bridge was $12 000 for nine 40-ft. spans, and the successful bidder offered a concrete bridge of six 56-ft. spans for $13 700.

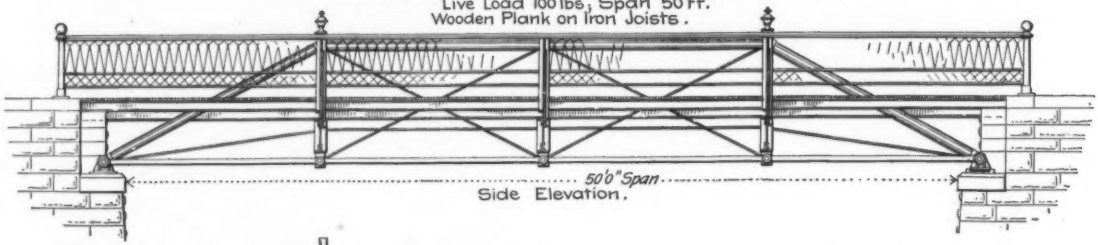
Mr. R. Wünsch states, in answer to an inquiry, that in Hungary a concrete bridge can be built as cheaply as a wooden one (with masonry pillars), and about 25% lower than an iron one.

This is, of course, not applicable to this country, because the labor here is far more expensive than there. But even if it should be possible to construct a concrete bridge for the same price as an iron bridge, it needs not a great prophet to predict a revolution in the construction of highway bridges, and the fact is that there is a general tendency to build highway bridges stronger and more independent of the live load.

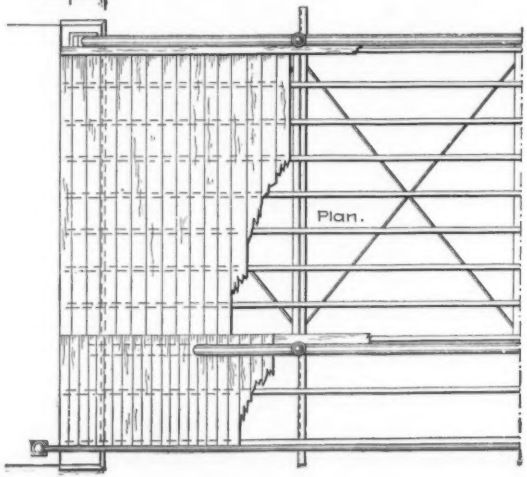
For purposes of comparison two comparative designs (Fig. 5 and Plates LX and LXI), one with 12-ft. span and heaviest loads (Fig. 5),

Ordinary Type of Highway Bridge (Class B.)

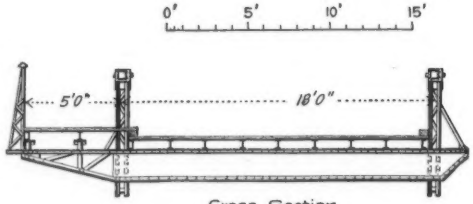
Live Load 100 lbs.; Span 50 Ft.
Wooden Plank on Iron Joists.



Side Elevation.



Plan.



Cross Section.

Weight of Truss: 14,300 lbs.
Floor Joists: 6,950 "
21,250 lbs.

Or 18·5 lbs. of Steel per sq. ft. Road and Railing.

Fr. von Emperger.
Consulting Engineer.

PLATE LX.
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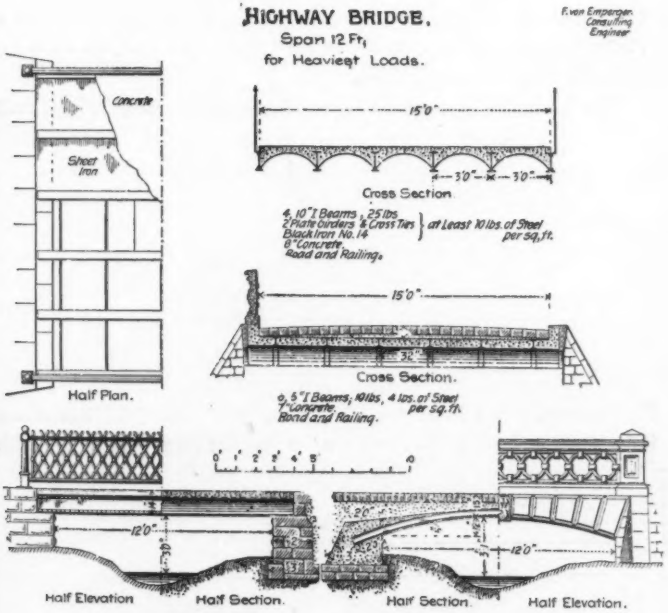
comparing the Melan type with a so-called concrete bridge, which, as the writer is informed, has been very often used in this country.

The weights of material are on each of the designs. They show the old design to be three times the more expensive.

The other designs (Plates LX and LXI) show a common iron bridge, which in the writer's opinion is in price about equal to the other design of a concrete bridge. The only uncommon part is the use of iron joists. But these iron bridges are constructed for a live load of 100 lbs. and less, without regard of heavier carloads, and hardly deserve the honor of being compared with permanent structures.

FIG. 5.

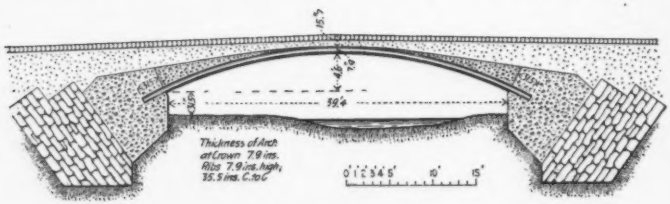


Everybody acquainted with current prices of these materials will be able to form his own judgment of the comparative cost of each type. The Melan system is the best, as it avoids any riveting in iron

and is, up to the present time, the cheapest as well as the strongest, as the tests have shown. This system is now in its second year, but the writer found it wise not to publish it before it had been thoroughly tested. Its largest field is its application to floors and vaults, to which a new system can be readily introduced, as they are not controlled by officials and depend more or less only upon the will of the owner.

It is now quite common in Europe to specify for buildings similar to office buildings, vaults with Monier arches. This is almost as common in Europe as the hollow brick floors (which are not used there at all) are here. Nearly 1 000 000 sq. ft. of the Melan type of floor have been built during the first year of its life, while of bridges the writer can only mention, besides those of 23⅛ ft. span near Neustadt, two others, both of which are here reproduced, to show two different types of construction.

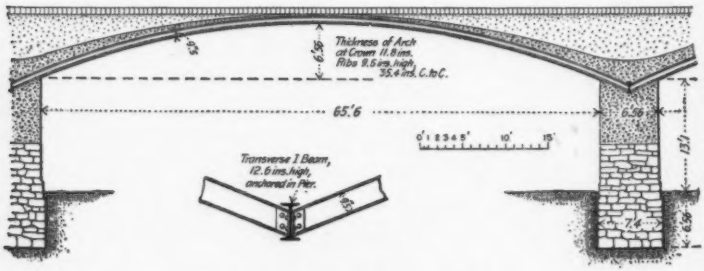
There is a bridge over the Struschka, near Oderberg (Fig. 6), with



Highway Bridge over the Struschka near Oderberg.

FIG. 6.

39¼ ft. span, which is a type of low abutments in which beams are imbedded; and a 65⅛ ft. span over the Moldau (Fig. 7) is a type with



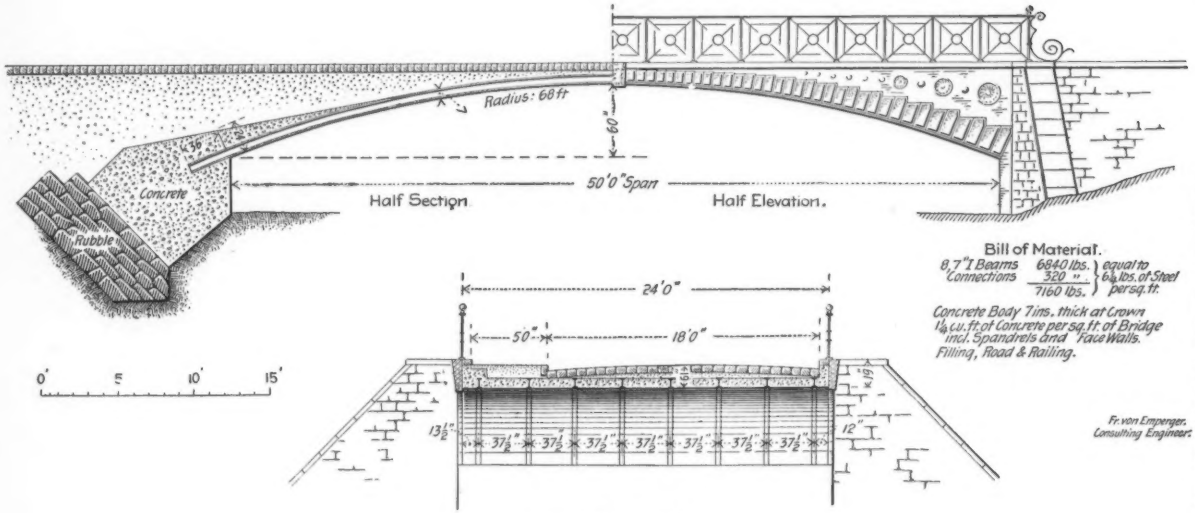
Bridge over the Moldau at Roetock,
6 Openings of 65.6 Ft.

FIG. 7.

Highway Bridge "System Melan"

Live Load 100 lbs, Span 50 Ft.

Ballast Floor



Cross Section at Apex.

Bill of Material.

8, 7" I Beams	6840 lbs.	}	equal to 6¾ lbs. of Steel per sq. ft.
Connections	320 "		
	7160 lbs.		

Concrete Body 7 ins. thick at Crown
1⅛ cu. ft. of Concrete per sq. ft. of Bridge
incl. Spandrels and Face Walls.
Filling, Road & Railing.

Fr. von Emperger.
Consulting Engineer.

PLATE LXI.
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high pillars and anchorage. There is still a third type with so-called hidden abutments, but there are as yet no structures built on this plan. These bridges were built by Pittel & Brausewetter, of Vienna.

Finally, some words about exterior finish. The cheapest is the concrete finish itself, with some fancy joints. The concrete can be colored by using chips and sand from colored stones, and Mr. Ransom gives them, with the use of wooden patterns, a stonelike appearance. Although Portland cement is largely used in this country for exterior finish in harbor work and for sidewalks, there is a general opinion that it will not weather well. The use of concrete containing lime (Coignet Béton or Roman cement) is largely responsible for this opinion. In Germany they imbed large stone in the face wall. The best material, which also gives for the smallest money the richest appearance, is terra-cotta.

The average span of concrete bridges now built is 50 ft., and never has exceeded 150 ft. in Europe, to the writer's knowledge. But there is no reason why a system, which has proven better than stone, should not be extended and reach the highest span in use with iron arches, as these bridges with the use of three lead joints and built shapes (instead of the rolled ones) are entirely equal to iron ones.

To accomplish this in Europe, there is needed, not only the right man and the right place, but also a third one, the right official, who will give his permit. Those three have not yet met. The writer regards the paternal care of the Government, so far as general specifications of bridges are concerned, as a good thing; but sometimes it is also an obstacle to progress, and on this account the engineers of this country have a good opportunity to surpass European structures.

DISCUSSION.

L. L. Buck, M. Am. Soc. C. E. (by letter).—I have followed discussions in technical journals relating to the Monier system of arches with a good deal of interest, and think the paper by Mr. von Emperger should possess value. The development of the system of arches of concrete must necessarily be largely based upon empirical information coupled with sound judgment and work executed with great care.

There are many places where a durable bridge of this character could be built with economy. The author's statement, that it is

claimed for this system in flat arches that the horizontal thrust at the abutment is almost entirely done away with, should be more fully explained. If the structure acts as an arch, this cannot be entirely true. If it has sufficient thickness at the crown, it may act as a beam; or, if it has depth at the haunches and is counterweighted on the other side of the abutment, it may act as a cantilever. Temporary apparent advantages may result from combining these principles, but it appears to me that it would generally be better to depend entirely upon the arch principle and use the iron, whether in the form of wire, or of rolled shapes, to insure the integrity of the concrete under extraordinary stresses.

The statement that concrete and iron expand the same under the action of heat is to me a surprise.

CORYDON T. PURDY (by letter).—I should like to call the attention of the Society to the fact that the use of steel in conjunction with concrete has already reached a considerable development in the construction of foundations for large buildings, in Chicago chiefly, and in other cities to some extent.

Mr. von Emperger's point, that these two materials are especially fit to be used together, will be appreciated by all of us who have believed this to be true and have shown our faith in that belief by our works. Ever since steel foundations, as they are generally termed, were first advocated, their enduring properties have been questioned, and even to-day some architects in that city will not use the combination. Every item of reliable evidence to show that iron or steel will not deteriorate in concrete, or *vice versa*, is therefore important in its relation to that construction.

A few words in regard to that work may not be out of place. I do not know who first conceived the idea, but it was first used by Mr. Burnham, and he well deserves the credit for its introduction. At first the metal was used only to supplement rubble masonry and concrete, then more dependence was put on the steel, and soon the masonry was omitted entirely and only enough concrete was used to give the metal a good bed and thoroughly cover and protect it. At first railroad rails were used, some of them old ones; then some beams were used, and, as soon as the iron pool was broken, beams were used almost entirely.

In rare instances riveted work has been used, and a few foundations have been made of light beams. Some thousands of tons of steel have thus been imbedded in concrete in these foundations.

Of course this is not the only solution of the foundation problem even in Chicago, and possibly not the best one, but the demand has been very pressing for something that would provide a large bearing without breaking the clay strata and yet be so thin in its vertical dimension that the basement room would not be destroyed. The steel

foundation was a cheap and natural solution of the problem. It quite completely fulfilled the requirements of that time; and if it will endure indefinitely, there are many good reasons why it may well be used for limited loads in all times.

Mr. von Emperger is very definite. He says: "Concrete is the best conservator of iron." The truth of this statement, for foundations or highway bridges or any other permanent construction for that matter, is fundamental and vitally important. If the perpetuity of iron or steel imbedded in concrete depends on conditions, it also becomes correspondingly important to have these conditions definitely fixed.

Jos. M. Wilson, M. Am. Soc. C. E. (by letter).—We have been using cement and iron or steel in combination for fire-proof floors since 1889 with great success. While not making any special claim for originality in the method employed, yet the exact arrangement adopted by us was a little different from anything that I had at that time seen elsewhere, and an illustration of it was published in one of the engineering papers in 1890.

The great desideratum in such cases is a system that shall be easy of construction without the employment of skilled mechanics and with a minimum of expense in the manufacture.

The form of construction adopted consists of the ordinary I beam girders, running from wall to wall, or from walls to columns, placed at distances apart of, say, from 10 to 18 ft., as may be required, between which are hung flat-iron or steel straps at intervals of 12 to 24 ins., the ends of each strap being bent and hooked over the top flanges of the girders, and the straps curved down, so that midway in their length they hang very close to the under surface of the intended floor. A level centering having been provided, a floor is built in of concrete, completely embedding the straps and the girders; or, if the latter are too deep for this, it is curved down on to the lower flanges of the girders. The plaster for ceiling finish is applied directly to the under surface and under the lower flanges of girders.

We have used these floors in the American Philosophical Society building, the Academy of Natural Sciences and the Drexel Institute, Philadelphia; in the Alumni building of the Rensselaer Polytechnic Institute at Troy, and are now placing them in a large training school in Indianapolis.

Those in Philadelphia were constructed of Portland cement. First, a coating on the centering of 1 in., formed of cement and sand, 1 to 2; then concrete, composed of 4 parts stone, to pass through a 2-in. ring, 2 parts sharp river sand, 1 part Portland cement; then a finishing coat on top of 1½ ins., 1 part cement to 2 parts sand, divided up into squares and tinted as may be required for the floor. If a wooden floor is desired, only 1 in. of finish is used, and nailing-strips, to which to nail the flooring boards, are bedded in light filling on top.

In the American Philosophical Society building, the spans for the first and second floors run from 8 to 16 ft.; for the former the straps being $\frac{3}{4}$ x $\frac{3}{8}$ in., 24 ins. to centers, with concrete 4 ins. thick, including finish; and for the latter the straps are $2\frac{1}{2}$ x $\frac{5}{8}$ ins., 24 ins. to centers, with the concrete 7 ins. thick. These floors were intended for 80 lbs. live load.

The third floor, of 16-ft. spans, carrying 210 lbs. loading to the square foot, being a library floor, has straps 2 x $\frac{5}{8}$ ins., 12 ins. apart, and concrete 8 ins. thick.

The Academy of Natural Sciences has five floors and roof, 17 to 18-ft. spans, all constructed in this way, the floors having 8 ins. of concrete, and straps $2\frac{1}{2}$ x $\frac{5}{8}$ ins., 18 ins. apart; live load, 100 lbs. to the square foot. The work has finished up beautifully without a sign of a crack or defect anywhere.

In the Indianapolis building there are about 67 000 sq. ft. of flooring to be laid. The concrete is to be of Louisville cement, 1 to 3 parts crushed stone with 1-in. finish on top of Dyckerhoff cement and sand, 1 to 2. The spans are about 11 ft., the straps or bars 1 x $\frac{1}{4}$ in., 12 ins. apart, and the concrete 7 ins. thick, lightened up with open cores running through between and parallel to the straps. This floor, which is to be constructed by a Cleveland firm, has been placed at so reasonable a figure as to compete successfully with wooden mill construction.

The Society may not be aware that a small concrete bridge was constructed last year over the Pennypack Creek, in Philadelphia, by the city authorities. The engineer, Mr. Carl A. Frick, states that it consists of two arches 25 ft. $4\frac{1}{2}$ ins. span each, with rise of 6 ft. 6 ins., and is built on a light skew: width out to out of parapets, 34 ft. The depth at crown is 2 ft. 3 ins.; the abutments, 7 ft. in thickness, and the pier, 6 ft. The spandrel walls, 3 ft. at the springing line, and 2 ft. at the coping. The entire work (foundations and all) has been constructed of concrete, German Portland cement, some portions being mixed of 1 part cement, 2 parts sand, 6 parts broken stone, and other portions of 1 part cement, 1 part sand, and 4 parts broken stone. Wire netting, $\frac{1}{4}$-in. wire, 12-in. mesh, was placed in each layer of the concrete.

It is stated by the engineer that a saving was effected in cost of about one-fourth over that of a stone and brick bridge. Without taking up an analytical consideration of the bridge, the depth of the arch appears to be much heavier than necessary for the span, and it is probable that with a closer approach to the vertical limits, still more saving in cost might have been effected.

Col. Wm. P. Craighill, Pres. Am. Soc. C. E.—There is one question that I would like to ask the author. On page 439 he says, "In this country, for instance, the use of concrete has been almost confined to floors." Is that intended to mean the use of concrete alone, or concrete in combinatian with iron or steel? The statement is very general.

Mr. VON EMPERGER.—I mean concrete and iron.

Col. CRAIGHILL.—The statement is so general that I felt called upon to take exception to it, for the reason that concrete has been used for various purposes in this country for many years. For instance, for more than a half century it has been used very extensively in the fortifications of the United States. One of the earliest treatises on the subject was by General Totten, once Chief of Engineers of the Army, printed in 1838.

FOSTER CROWELL, M. Am. Soc. C. E.—I should like to ask the author more definitely in regard to the expansion rates of iron and concrete. The reference in his paper seems to apply only to experiments upon Portland cement and iron, in which the rate is about the same. It seems to me that the expansion of a material like concrete must be as variable as the composition of the concrete. There have been quite a number of experiments made with reference to concrete as a composite material, and not only to cement. Mr. Buck has raised the question, and it seems to me that it may not be correct to assume that all concrete would expand equally with iron.

GEORGE HILL, M. Am. Soc. C. E. (by letter).—The various methods suggested for the spanning of openings by means of systems less expensive than those in use at the present time, while equally enduring and strong, have been followed with much interest by engineers. We are therefore much indebted to Mr. von Emperger for the very interesting description of the Melan system.

So far as its use for highway bridges is concerned, I refrain from expressing an opinion, since that is outside my line. We are told, however, that it is coming into extensive use for floors in Germany, and it is intimated that it would have an extensive use in that direction in this country; and it is concerning this that I wish to speak.

It is true that the actual area of floors laid in it so far is relatively small as compared with the use made of the better known styles of fire-proof floor construction in this country, but it is very much in excess of any record which we have to show for the first year after the introduction of any of our materials, and it therefore behooves us to examine the system with a great deal of care before expressing an opinion. Analyzing the arch, we find the compression is in part taken up by the concrete, and in part by the curved metal ribs; that the thrust is absorbed by the abutments, as shown by the drawings, or presumably by tie-rods in building construction, and that we further cause the concrete to act as a beam between the curved ribs, and compel it to do all of the work of which it is capable in that position. We must use the concrete in this latter way, and it is not good engineering to add to this duty an additional one by making it act as an arch, thus putting the fibers under a maximum strain in two directions. Mr. von Emperger cites tests, also, which show a bending moment at the haunches which

is apparently far beyond the strength of the concrete to resist, acting as an arch, but which is readily carried by it to the curved ribs and by them resisted. Taking this view, all the equations except 9, 10 and 11 are cleared of their complexity, and we should simply calculate the concrete as a beam between the curved ribs, establishing proper ratios at once for span and depths, and calculate the ribs for the strains which are imposed by the live and dead loads acting as arches. Under this supposition, we would have the same amount of metal required as in a beam, minus the web.

In considering the use of such a flooring we have:

First.—The use of curved ribs which require careful mill work, and, in order to secure the results which are aimed at by the author of the paper, a close union between the concrete and the iron, all the metal must be free from rust until it is used in the building.

Second.—The cement must be of uniformly high grade.

Third.—The workmanship must be honest and such as to insure a proper use of the materials.

Fourth.—In at least 50% of the cases a suspended ceiling must be provided, so as to make the bottom surfaces absolutely flat.

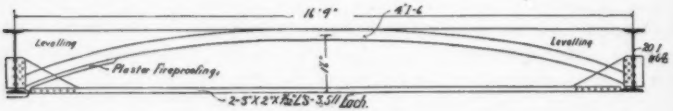


Fig. 1.

Suppose that we had an area 16 ft. 9 ins. on one side by 20 ft. on the other side to cover. By the Melan system, we would put in curved ribs, assuming 5 ft. spacing on centers, spanning the narrow way, and using the construction shown in Fig. 1, putting in tie-rods; then we would have:

COMPARATIVE TABLE.

Melan System—

Arch ribs. Area $= \frac{5 \times 175 \times 16.75 \times 16.75}{8 \times 1.5 \times 12\,500} = 1.64$ sq. ins. $=$ 4-in. I beams, 6 lbs. per foot.

Tension Member. Area the same; use two 3 x 2 x $\frac{7}{32}$-in. angles, 3.5 lbs. per foot each.

Weight per linear foot, 13 lbs. $=$ 2.6 lbs. per square foot.

Girder. $\frac{21\,975 \times 82}{16\,000} = 112.5 = R$, and requires 20-in. I beams, 64 lbs. per foot.

Weight on girder, 1 280 lbs.; area supported, 335 sq. ft.; weight per square foot $=$ 3.82 lbs.

Connections. Plates, 30.5 lbs.; angles, 21 lbs.; rivets, 10 lbs.; Total, 61.5 lbs., over an area of 83.75 sq. ft. = 0.74 lbs. per square foot.

Total metal = 2.60 + 3.82 + 0.74 = 7.16 lbs. per square foot.

Flat Arches—

Floor beams, 12-in. I beams, 40 lbs. per foot; 5 ft. 7 ins. centers,

$$\frac{20 \times 40}{20 \times 5.58} = \ldots\ldots\ldots\ldots\ldots\ldots \text{ 7.17 lbs. per square foot.}$$

Girders, 15-in. I beams, 60 lbs. per foot

$$\frac{60 \times 16.75}{335} = \ldots\ldots\ldots\ldots\ldots\ldots \text{ 3.00 " "}$$

Tie rods and connections, allow..... 1.27 " "

Total metal..... 11.44 " "

QUANTITIES AND PRICES PER SQUARE FOOT.

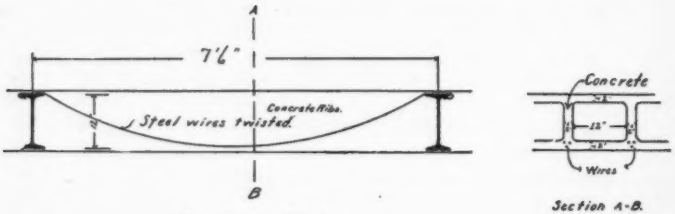
MELAN ARCH.	CENTS.	FLAT ARCH.	CENTS.
Metal, 7.16 lbs. at 35 cents.....	25.06	Metal, 11.44 lbs. at 3 cents.....	34.32
Arch concrete, 6 ins. at 35 cents.....	35.00	12-in. arch blocks.....	26.00
Leveling concrete, 4 ins. at 15 cents...	15.00	½-in. leveling.....	5.00
Depth, 22 ins. at 2 cents.....	44.00	Depth, 16 ins. at 2 cents.....	32.00
Total.....	119.06	Total.....	97.32
Weight per square foot.....107 lbs.		Weight per square foot.....69.44 lbs.	

Flat arch saves per square foot in weight, 37.56 pounds.
" " " cost, 21.74 cents.

Clearly there is too great a difference in cost under American conditions to make the arch of general application, even where we entirely neglect the cost of the suspended ceiling. Of course there may be special cases where it would be exactly the thing, but the general case is as given above. In addition to these difficulties, there are those existing in the actual construction, which those who have devoted their entire attention to the engineering of building, know to be almost insurmountable. These are: first, with regard to the proper use of cement; second, the time which must elapse before loads can be applied; third, the lack of fireproof qualities. In engineering structures we have an inspector, thoroughly competent, constantly upon the work to see that every portion of the mortar is used under the most favorable conditions. In buildings this cannot be done, and reliance therefore, must be had, to a greater or less extent, on the knowledge of the men, who, with the most honest intentions, constantly use re-tempered mortar, mortar that has been far too highly dosed with sand,

or contains at least 50% too much water. As a consequence of these conditions, we have established the general rule that no cement should be used in tension, and that the tensile strength of the cement should not be relied on, which, if followed, would at once bar out this system.

This is said with a full knowledge of the work done by Mr. Wilson and others, and with the further knowledge that the concrete arches in use here generally only rely on the compressive strength of the concrete and are so lightly loaded under usual conditions that well-dried mud would do equally well, if given a good wearing surface, until the crucial test came. Thus assume that the section of 1 ft. is as in



Elevation Fig. 2

Fig. No. 2, we should have 64 sq. ins. in it. Assume that the weight is 60 lbs. per square foot, and that the live load is 30 lbs. per square foot, which is slightly above the average of the Boston determinations, there we would have a compression on the total area of 49.31 lbs. per square inch, which is very light. Even if we treble the live load we should only have 82.05 lbs. per square inch. This shows how great the difference is between the use of cement in the Melan system and our present use of it, and also explains why floors so built do not fail in many cases. Were we able to get thoroughly first-class concrete for floor arches we could adopt the principle described by Mr. Wilson in a more economical form by using steel wire of double the strength of the plates he uses and save probably 50% of the weight of the floor framing (see Fig. No. 2). Companies have been formed to exploit such a construction, but they have not had marked success.

We must also remember that the general use is the one to be considered, and that the building departments of the various cities in this country have had so sad an experience when trusting to the honesty of the builders that they condemn any system which is not of the simplest character and which depends on so many elements for its success.

In a country where eight months is sufficient to practically complete a building costing nearly $1 000 000, the statement that nearly three

months must elapse before the arches can be fully loaded would of itself bar them out, since often the heaviest load an arch is called on to sustain is that due to the storage of materials, and is applied before the arch is five days old and after the centering is removed.

We know that cement has been subjected to intense heat in the process of manufacture, but we also know that when heated again parts are apt to fly off, and especially where the cement is thin, as under the flanges of the beams, where it is most needed and where it would certainly fly off if touched with a stream of water while hot, thus leaving the beam unprotected against subsequent attack.

Concerning the relative strength of the two forms when used for floor construction there is no question in my mind but that the flat arch floor is the stronger under all similar conditions of treatment, since it uses mortar in compression only; can be laid up without mortar if desired; utilizes each material in the best possible way, and in such a way that long-continued vibration cannot in any way injure it. The question of relative strength is one that we are likely to disagree upon for some time still, but, briefly summarizing the tests, we know that the fire clay alone will carry about 2 000 lbs. per square foot, in compression; that the only tests to destruction of flat arches showed that the skewbacks were very weak, shearing off under about 1 000 lbs. per linear foot; that properly designed skewbacks have carried as high as 5 000 lbs. per linear foot without failure. That in the ordinary flat arch there is an area transmitting compression of about 30 sq. ins., and a joint area of about 144 sq. ins. in the comparative case given, and therefore we should have a safe uniform load on this 12-in. arch of (safety factor, 10):

$$L = \frac{W}{5.58} = \frac{8 \times 30 \times 8 \times 200}{67 \times 5.58} = \frac{384\,000}{373.86} = 1\,027.$$

This is on the assumption that the whole area of the arch acts; if only three-quarters acts, then the load becomes 770 lbs. per square foot. This uses only well-known formulas, and a conservative value of the crushing strength. With the Melan system, we must assume the extreme fiber strain at 90 lbs. per square inch, in order to obtain a reasonable thickness of slab between arch members and this is confessedly very high (see page 449).

We cannot determine from the test given what the concrete of the Melan arch would carry, since failure was occasioned by a load concentrated on the metal rib and concrete at one point, and when the rib bent the cement ruptured in every direction (see Plate LIX). The author states, page 453, that 640 lbs. gives a factor of 7.1. Whereas, with the flat arch, according to our figuring, the factor is 44. There is no doubt but that we need an exhaustive series of tests to determine what the strength of our flat-arch floors really amounts to.

Concerning the reference by Mr. Purdy to the use of cements and steel in Chicago, I would refer you to the *Engineering News*, pages 116, 265, 312, 415, Vol. XXVI, which quite clearly demonstrates to me that this use was not good engineering. Both then and now, it would cost more than a combination of girders designed according to the generally accepted theories of flexure.

JOSEPH M. WILSON, Vice-President Am. Soc. C. E.—I think I stated clearly that I made no special claim to great originality in my form of construction. I am aware that construction of a general character similar to what we have adopted has been used in San Francisco and perhaps in other places, the detailed arrangement in our case being different. Some of the floors we have built have been finished now for four or five years.

It is possible that the building laws of the city of New York may prevent the use of floors constructed as these are, but I do not think that one of the reasons mentioned, to wit, the question of the builder not doing his work properly, should be an excuse for its rejection.

So far as plastering is concerned, the ceilings were plastered directly on the cement, and there has been no staining or other trouble with them. The ceilings in the extension to the Academy of Natural Sciences in Philadelphia could not look better. They can be considered a perfect piece of work, although they have been exposed all winter without heat in the building, as no boilers have yet been provided, and the building has been up since last fall. The work could not be in better condition than these ceilings are.

I think also the question of expansion and contraction is somewhat of a bugbear. In buildings the conditions are a little different from those of outside work. The variations of temperature are never very great, as the walls protect the interior from cold in winter and heat in summer, and in winter there is always artificial heat when the buildings are occupied. I do not think the extremes are very great in any of our buildings.

W. G. TRIEST, Jun. Am. Soc. C. E.—The span of 150 ft. has been mentioned as the maximum which has been used for concrete bridges. By concrete bridges we must understand iron-concrete bridges here, but it will not be inopportune to call attention to a concrete-arch bridge of 164-ft. span, which has recently been built over the Danube at Munderkingen, Wurtemberg. This bridge is the more interesting, as, in a sense, it is a combination bridge, also having some iron members which have mostly been used in iron construction only. The rise of the arch is 1 in 10, and its width, 24 ft. 3 ins. No spandril filling was used in order to keep down the dead load. At the apex and at the skewbacks cast-iron half cylinder and socket joints were inserted, in order to render harmless the effects of the settling. The arch settled $2\frac{3}{4}$ ins., when the centers were struck, and $1\frac{5}{8}$ ins.

more when the roadbed was put on. The skewback joints were then covered with concrete, while the apex joint remained free to allow of a movement due to changes of temperature.

An instructive proof of the preserving power of concrete was furnished last year. The cast-iron concrete-filled pillars of a viaduct had been taken down. When they were broken up, a wrench was found inside of the concrete, having been buried therein for 22 years. It had kept a metallic blank surface under the concrete, while part of the wrench that had been imbedded in coal ashes had suffered greatly by rust.

EDGAR B. GOSLING, Jun. Am. Soc. C. E.—I have made large concrete blocks which, after long life, show perfectly smooth and good surfaces, and I therefore do not agree with Mr. O'Rourke that the cause of peeling or shelling off at the surface is due to differences of expansion and contraction in the interior, and on the surface of the stone. To further bear out this point, I would state that I have had considerable practice in the manufacture of artificial stone, having, among other work, put up a structure about 180 x 200 ft., entirely walls, etc., of Béton Coignet. This structure had columns 15½ ft. in height by 18 ins. in diameter, in one piece, and I have found that the surfaces are perfect, and that expansion and contraction have not the effect mentioned on well-made and properly rammed concrete or béton. I should like to take exception to the remark made by Mr. von Emperger, on page 457 of his paper, in which he states that "Coignet Béton is largely responsible for the general opinion that concrete will not weather well"; as this statement (although very handy to cover the many drawbacks of the Melan system) is by no means borne out by facts.

CARL GAYLER, M. Am. Soc. C. E. (by letter).—The use of Portland cement concrete has wrought a revolution in all branches of civil engineering, and it seems that we are only in the beginning of the radical changes, which in bridge work, sewers, water works, railroads, etc., are following its introduction. Any new form, therefore, under which this material can be utilized is worthy of the closest attention, and this is what gives the paper under discussion an unusual value.

The shallow and exceedingly thin concrete arches which have of late years been built in Germany and Austria have excited a good deal of surprise on this side of the Atlantic. They are, undoubtedly, the most convincing proofs of the wonderful qualities of the Portland cement, of which the Germans, judging from papers read in Chicago, are justly proud. Considered as advertisements of German Portland cement alone, they seem to be worth the money which they cost. They have, so far, not been imitated in this country, although we cannot be accused of being slow in appreciating and appropriating for our own use any new valuable features in engineering from other parts of the

world, and it is very likely that, whenever such arches come into general use, their proportions will be considerably modified; to rely on concrete to resist tensile strains when it is obvious that the unavoidable settlement, after removal of the centers, may start a crack at any place, appears like unsound engineering. With all due respect for the minds who planned and executed these bold arches, the question may well be asked whether an engineer is justified in reducing the dimensions of a bridge to such evidently dangerous limits; as, for instance, 128-ft., and 150-ft. spans with a thickness of crown of $6\frac{2}{3}$ ins., any more than the bridge engineer, who, in designing an iron bridge, strains the metal close to the elastic limit. It looks as if the whole talent and ingenuity of these engineers had been employed on the problem of building the thinnest possible arches with a material estimated in value as one of the precious metals.

The fact that steel nettings and steel frames of various designs, imbedded in the concrete, are now being used for such arches, seems to prove that the European engineers themselves are not without their misgivings as to the safety of these structures, and the author of the paper discusses three different methods now in use in Germany, Austro-Hungary and Switzerland.

The oldest (comparatively) of these, the so-called Monier system, has been known for some time. It will probably never be used for spans exceeding 50 ft., as it admits in nowise of a calculation of its strength, the engineer having to be guided entirely by the results of tests made on test arches. The second system, introduced by Mr. Wünsch, of Buda-Pesth, is a great improvement over the former, as it admits of a more intelligent proportioning of the metal work. It is not stated quite clearly in the paper whether the iron frames are designed so as to resist the loads by themselves, the concrete only acting as spandril and lateral bracing, but we can infer this from Mr. von Emperger's remark after speaking of the failure of one of the Monier arches, "that this difficulty, *i. e.*, unsafety of the arch owing to the use of worthless concrete, has been overcome by the other systems now in use." The Wünsch system is also superior to the last of the three systems (Melan) because the tensile strains of both the upper and under sides of the arch are resisted by metal.

For the system of Professor Melan, to which Mr. von Emperger gives the preference, the claim is made, supported by a calculation, that the concrete and the beams in the arch work together. The full strength of the concrete, in addition to the strength of the steel beams, is counted upon as acting as an arch, and at the same time as a very efficient lateral and vertical bracing. The claim is made, for the first time in the history of civil engineering, that iron and stone, in spite of their different strength, elasticity and thermic expansion, if properly proportioned, act together like one material. The case is entirely

different from the use of anchor bolts in a bridge pier, because there the metal is held only at the ends and is stretched and released without account being taken of the contact with the masonry; it is also different from the use of steel beams with concrete arches between them, because the steel beams and the arches each carry their own share of the loads individually, but we would have a case similar to what is claimed for the Melan system if we would design a column or pillar of steel and concrete assuming the strength of the pillar to be equal to the sum of the strength of the steel and concrete.

In regard to those properties of iron and cement which, in Mr. von Emperger's opinion, make both materials "especially fit to work together," I wish to say:

1. We know that the modulus of elasticity of different kinds of steel shows little variation, but we also know, and we can see from the paper under discussion, that the modulus of elasticity of Portland cement concrete is a quantity about which we know very little, the 750 000 assumed in the calculation of Mr. Melan is about one-half of the figure we have been in the habit of assuming, and is obtained from experiments which have not been published yet.

3. I agree with the author that concrete is the best preserver of iron, but the fact that the underside of the steel beams is not protected is objectionable in a system of highway bridges for which no expense of maintenance is claimed.

4. The remarkable fact, given on the authority of Mr. Bonniceau, that the thermic expansion of Portland cement (should this read Portland cement concrete?) is within $\frac{2}{10\,000\,000}$ parts the same for both materials for 1° Celsius may, for want of sufficient data, go unchallenged, but surely, steel being a better conductor of heat than concrete, there must be considerable difference, at the same moment, in the expansion of the two materials. The influence of temperature would, in view of the exceedingly rapid changes of temperature in this country be far greater here than in European countries.

I am willing to acknowledge that for short spans, and in cases where a graceful appearance of the structure is desirable, the Melan system has its advantages; but I believe that the European official with whom Mr. von Emperger finds fault for once acts wisely in limiting the lengths of the spans.

BERNT BERGER, Assoc. M. Am. Soc. C. E.—The description of the different systems of concrete iron construction in Mr. von Emperger's paper is very interesting, and the tests enumerated would apparently point to the possibility of wide application of the systems to bridge construction.

There are, however, some points in the paper which have not been made quite clear, and in other places the author has made more sweep-

ing claims for this kind of construction than seem warranted by an unbiased study of the tests described.

The structures are treated as arches. In the description of the Wünsch system arch it is stated that no horizontal motion of the abutments was measurable, and that the horizontal thrust was changed to a lift. No motion may have been measurable, but that the thrust must have been there is readily judged from an inspection of Fig. 3 in the text. Evidently it acted more as a plain beam and a cantilever combined than as an arch. In a cantilever we have, however, a horizontal thrust to take care of, and in this case the abutments were strong enough to resist it. How columns would have acted if the "arch" had been put on top of them, as suggested, is a different thing.

The author's claims of superiority, for the Melan system especially, suffer somewhat by such statements, as "it is not difficult to show that these bent beams would safely carry alone the load intended," and, further down, in comparing the three systems described: "It must be remembered, first, that any of them will do, because both materials will do it as well alone."

We have as yet no actual proof that concrete and iron will act so well together as the author claims. His description of the tests made in Austria does not prove that they act so well together in actual service, and for any length of time. The application of the combination, for instance, to building foundations, especially in Chicago, has been followed with a great deal of interest, and doubt as well, and developments, in one direction or another, will be eagerly watched; but the method is of so recent origin that no reliable conclusions can as yet be drawn. The use on the Pacific coast of beams and girders of concrete, with iron rods imbedded, to take tensile strains, seems to have met with some degree of success. It would be interesting in this connection, to hear of observations made where such girders have been used for sidewalk vaults, as I understand they have, and exposed to variable loading. This form of the combination has, however, the advantage of having the iron rods so placed as to make the strains which come upon them more clearly defined, and no reversal of strains takes place. It was also found advantageous to use twisted rods in order to give the concrete a better grip upon them.

The tests of concrete-iron arches, described by Mr. von Emperger, certainly show favorable results. But it must be remembered that these arches were built for the express purpose of being tested. They had been given a certain time, varying in the different cases, to settle and harden, but none of them had been in actual use to carry traffic. The author claims for this system of construction freedom from vibrations, a rather bold claim, especially for arches of so slender dimensions. A question is, whether this kind of an arch would not, after having been exposed to the traffic intended for it for some years, have

suffered so much from just such vibrations as to seriously impair that glue-like bond.

Another and even more serious source of danger to that bond would be the effect of changes of temperature. Even if Portland cement has the same thermic expansion as iron, that by no means proves that concrete has. A mass, consisting to such a large extent of stone as concrete, would rather be supposed to behave like stone, and we know that stone and iron are not alike in this respect. Besides, the thermic conductivity of iron is much greater than that of concrete. Under changes of temperature the iron will be heated or cooled much quicker, and to a far greater extent, with a corresponding greater expansion or contraction, than the neighboring concrete, if not imbedded sufficiently deep, and the illustrations in the paper show the iron ribs practically exposed. That the result would be disastrous to the bond seems clear.

I think, therefore, that the Melan system cannot be considered thoroughly tested before more light has been thrown on these points. If a test could be made of an arch, which had been in use for some years, it might give us some valuable information in this respect, but until such information can be had, it would seem only proper to move cautiously in adopting the proposed system.

There appears to be no reason why concrete filling cannot be used between the iron or steel ribs of an arch instead of the ordinary floor construction, if it is thought desirable, and it would, besides, do useful service as a continuous stiffener of the ribs. Whether it would be economical to use concrete in this way is another question. It would, however, seem advisable to introduce iron rods between the ribs, to prevent them from spreading, and not trust to the rather uncertain adhesion between the two materials.

The advantage due to steel having a modulus of elasticity about 40 times higher than that of concrete is not so manifest. Limiting the tensile strain on concrete to 42 lbs. per square inch, as introduced by Mr. von Emperger in the calculations, the tensile strain on mild steel should not exceed $40 \times 42 = 1\,680$ lbs. per square inch, if the elastic behavior of the two materials should be alike.

The tensile strength of concrete we know comparatively little about. The author makes the statement, that it can be taken from 290 lbs. to 340 lbs. per square inch, and infers that these figures may be doubled in determining the transverse strength of concrete beams. These figures seem, however, very high. General Gillmore, in his "Practical Treatise on Limes and Hydraulic Cements," gives the results of experiments made on the transverse strength of concrete beams at Boulogne-sur-Mer, in 1858. The *Engineering News*, of May 17th, 1890, reprinted from an Austrian technical journal the results of a series of transverse tests of concrete beams, made at Ymuiden, Holland, in 1888–89.

In both these places, the tensile strength of concrete, determined by transverse tests, is found very much below the figures given by Mr. von Emperger.

In the calculation of concrete-iron arches given in the paper, no attention is paid to temperature strains, which, in fixed arches of so little rise as those shown, may add considerably to the strains due to dead and live loads, while the difference in thermic conductivity will further induce secondary stresses.

Everything considered, it would seem that we ought to be very conservative in proportioning concrete sections, for tensile strains, especially in combination with iron or steel.

In this connection, it should also be borne in mind that in proposing new forms of construction, a fair comparison with present forms calls for equally safe strains in the structures to be compared. Claims of economy of material should not be based on the introduction of higher unit strains. Such economy can be gained as well in our present forms of construction.

Finally, it has not been made clear how an arch on the Melan system can be made more tornado and high-water proof than any present form of well-designed bridge.

W. R. Hutton, M. Am. Soc. C. E. (by letter).—Some of the statements on page 440 of this paper are inaccurate, and should not go into the *Transactions* of our Society without correction, although they do not directly affect the matter under discussion.

The arch over the nave of St. Peter's is a semicircle (not a parabola) of 90-ft. span. The top of the imposts is 110 ft. above the pavement, making the total height under the key 155 ft.

Pont d'Alma is not a railway viaduct, but a very handsome street-bridge in Paris. The rise of its oval arches is one-fifth of their span. It is not built of concrete, but of dressed stone backed with rubble masonry. Pont Napoléon, which carries a city street as well as a railway, also within the limits of Paris, is of similar, though less luxurious, construction. Notes of the German bridges referred to are not at hand.

If the concrete arch of 150-ft. span was made with a hinge at the crown, the depth of $6\frac{3}{4}$ ins. at that point is not extraordinary. If otherwise, some details of its construction would be of interest.

The Wünsch system described in the paper is the same in principle as the Cadiat system for iron arches at one time much in vogue on the Continent of Europe. One who has studied the system remarks concerning it, that the top chord or stringer is liable to excessive temperature strains which may easily exceed the elastic limit of the material. This actually occurred at the Pont d'Arcole in Paris, in which there is a perceptible dislocation at the center, at which point the rib is but 16 ins. deep. These strains would be relieved, and others not

increased by cutting it in two at the center. A second defect of the system is the racking of the piers by the alternate thrusts and pulls due to local loads and changes of temperature. The abutments of the bridge of Szegedin are said to be dilapidated from this cause.

The use of iron in combination with masonry to supply the want of tensile strength in the latter is not new. Indeed, one-half of a Wünsch "arch" of 120-ft. span and about 11-ft. rise, cut in two at the middle, would represent Brunel's model half-arch, built of brick and hoop iron about 50 years ago.

As to the so-called Monier system the principle is correct, to build into the concrete iron bars or straps at points where tensile stresses come, with the condition that the metal shall be used in such form and manner that its value can be ascertained. The concrete floor described by Mr. Wilson fulfills this condition.

The Melan system appears to be an entirely different thing. In it no attempt is made to supply directly with metal the deficiency in tensile strength of concrete. It is a combination in one structure of materials entirely distinct in their characteristics, in which combination the moment of inertia of the sections is inversely proportioned to the modulus of elasticity of each material.

It is difficult to realize that such combinations would result in sections so much more slender than could be adopted with either material used alone.

The theoretical investigation of the examples given would involve a labor which cannot now be devoted to the subject.

It may, however, be concluded that in the hands of one familiar with the theory of the elastic arch, and thoroughly acquainted with the materials used, some economies may be realized—economies which in unskillful hands may be too dearly bought.

GEORGE A. JUST, M. Am. Soc. C. E.—In reply to the inquiry of Mr. Crowell, it may be stated that Mr. Hyatt in his book (which the Chairman announces Mr. Worthen has sent us to-night) proves the extension of both materials to be equal, in iron-concrete beams, whether under load or fire, at least to his own satisfaction, and concludes that, "in combining concrete with iron for building purposes, the two materials may be regarded as practically homogeneous."

The application to structures, however, of iron in various shapes in combination with concrete, must very much antedate the period fixed by the author of the paper. To trace the origin of this device would be much like tracing back the origin of the telephone or the first use of the cantilever principle in bridge work.

In fact the book referred to by our Chairman was published in London in 1877, for private circulation, by Thaddeus Hyatt, with the rather lengthy title: "An Account of Some Experiments with Portland Cement Concrete, Combined with Iron, as a Building Material

with Reference to Economy of Metal in Construction, and for Security against Fire in the Making of Roofs, Floors and Walking Surfaces."

Mr. Hyatt first quotes from Mr. Fairbairn's work, "On the Application of Cast and Wrought Iron to Building Purposes," and shows that even at that time two systems had been introduced in France styled "Système Vaux" and "Système Thuasné." In the former the iron of the combination consisted of wrought plates bound together by tie rods, which are crossed by other rods supporting the ceiling. In the latter system rolled iron joists were used instead of plates. Mr. Hyatt also mentions his factory building in Farringdon Road, London, as being constructed on similar lines, and to win favor for his method had about fifty of such combination beams tested to destruction by Mr. Kirkaldy. Mr. Hyatt also prints a letter addressed to him by that eminent American experimenter, R. G. Hatfield, showing that the former made and had tested combination beams as early as 1855.

From his investigations he concludes that the method of combining iron and steel with concrete, "should be applicable also to bridge construction," and urges its use in the constru tion of chimneys, lighthouses, stairways, etc.

It may be added that in 1883 there was published in the papers of the American Society of Mechanical Engineers an article by Mr. W. E. Ward, entitled "Béton in Combination with Iron as a Building Material," wherein he describes the construction of a dwelling at Port Chester, N. Y., where "not only the external and internal walls, cornices and towers," were constructed of béton, "but all of the beams, floors and roofs were exclusively made of béton, re-enforced with light iron beams and rods," built by him in 1875. So much for the history or antiquity of this method of construction.

Since the author of the paper states that this system finds, "its largest field in its application to floors and vaults," it is perhaps permissible to touch upon its use in such constructions. He is, however, in error when he states that with us its introduction is "not controlled by officials," and that the paternal care of government is entirely lacking, requiring no special permit or supervision.

In illustration: The law governing building operations in New York City says all arches placed between iron or steel floor beams shall be of brick or stone, or "sectional hollow brick of hard burnt clay, porous terra-cotta, or some equally good fire-proof material." Under the latter clause a special application must be made for the use of plain or combined concrete.

Such applications have heretofore generally been denied by those in authority, because the safety of such floors depends so much on the honesty of the contractor, the care of the mechanic, the action of frost, and the not too early removal of the centering.

When it is borne in mind that such official inspections are, after all, often only superficial, not to say dishonest, and that the author himself records disasters by dishonest work, or too early removal of centers, then the grounds of refusal given do not seem entirely faulty or unjust.

There now exist in this market companies prepared to execute work of this composite character, and some of the methods employed, notably those in which single wires are passed continuously over the upper flanges of the beams, and depressed between the bays to within 1 in. or so of the ceiling line by lengths of gas pipes, the whole being filled in with cement concrete, imbedding both beams and wires, have advantages not possessed by any of the systems set forth in the paper and would no doubt prove keen competitors with the same, both from technical and economic points of view.

In making a comparison of bridges built in this manner with those built entirely of iron or steel, it would seem that aside from the questions of relative cost and strength, the former should suffer, if, as the author states, they will not have developed their full strength "before three months," for generally bridges are not built until there is an immediate demand for their use.

When the author speaks of a system that changes a "horizontal thrust to a lift," or when he speaks of "concrete as inelastic, like cast iron," he is not altogether clear. Cast iron is certainly elastic, even if more so than concrete, and the elimination of thrust in an arch means a violation of the accepted principles of statics.

J. F. O'ROURKE, M. Am. Soc. C. E.—It seems to me, from reading this paper, that the writer of it advocates the use of concrete arches with iron beams. There have been several objections made to this to-night; that there is an uncertainty as to what the character of the concrete might be, whether it would be good concrete, or whether the contractor would be honest. I claim that if there is any building material in general employment in this country it is concrete, and it is coming into use more and more all the time, so we ought not to consider so much whether you can get good concrete as whether good concrete would answer. The reason why we have not adopted concrete masonry without stone facing in this part of the country, where we have cold weather, is that the concrete does not hold together on the surface. The reason is very plain. The expansion and contraction of concrete is about the same as iron, but it only heats on the surface; the result is that you have a shell that is expanding and contracting, with daily variations of temperature, while the interior of it maintains its temperature practically unchanged except from season to season. I cannot recollect a single instance of concrete *en masse* around New York in which that effect has not appeared. Sound it when new, and it is like stone. In the course of time, the surface be-

comes loose and shells off. Another layer loosens and falls off in the same way, and thus the process of disintegration goes on. Generally speaking, an arch made of concrete will eventually disintegrate in this climate. When you put it in conjunction with iron, you introduce another element of disintegration. Any engineer who has used heavy masonry and heavy iron construction upon it knows that the masonry expands and contracts to a hardly appreciable extent in comparison with the metal. It is plain that if you put an iron beam in concrete, with the flanges flush with the surface, that the beam will not be of the same temperature as the concrete, when the changes in temperature are taking place rapidly. We know that the iron will creep in and out of the concrete. The fact that such structures are not generally adopted in this country is because this is fully understood.

I think, also, that there is a great deal to be said on the subject of concrete. There are wide differences of opinion as to how it should be mixed. The author says it ought to be rammed in in 6-in. layers. To do this it must be more or less dry. Some years ago I had occasion, for a certain bridge in the South, to build all concrete piers, and when the boxing was removed from the first of them, the surface was rough; on the surface, for the remaining piers, the concrete was mixed so wet that it puddled itself, gave smooth surfaces, and was as dense as if rammed. It may be stated, as a general principle, that concrete takes up the amount of water necessary, rejecting the surplus before setting.

This matter of concrete, I think, has not been sufficiently discussed among engineers, and it is very desirable to have any paper that brings out a discussion on the subject.

It is fair enough to say, in conclusion, that I do not believe that there is any likelihood that the general practice will be departed from and that we shall have concrete built here without stone facing above the ground, excepting blocks whose sizes permit of this surface motion within the elastic limit.

JOHN BOGART, M. Am. Soc. C. E.—There are some excellent examples of concrete construction in some of the hydraulic concrete pavements which have been laid in New York and in many other places. Those which have been properly made have continued in good condition for many years under circumstances of extreme exposure and constant wear. The pavements of Broadway and of Chambers Street, adjacent to the City Hall Park, of New York, were laid twenty years ago. They were laid under carefully prepared specifications, and they are now in good condition notwithstanding the enormous travel of pedestrians which has been constantly upon them. There are other pavements which seemed, when laid, to be of similar construction, but which failed after short service. The difference was in the materials used and in their proportions and manipulation.

BERNARD R. GREEN, M. Am. Soc. C. E. (by letter).—Mr. von Emperger's collection of examples of thin concrete arches reinforced with wrought iron in some of its many new and specially adapted forms is a valuable contribution. It will increase the confidence of engineers in the utility, safety and economy of cement mortar and concrete when intelligently and skilfully handled. Only those of wide experience, however, in the personal testing and manipulation of cement, sands and ballast of several kinds and qualities, are likely to fully appreciate the real nature of concrete. It is so largely a matter of experience and skilful, honest manipulation that the successful attainment of higher results has not infrequently been the reward of the workman rather than of the theorist, as in the case of Jean Monier. Such manipulation will always give satisfactory results even with very ordinary materials, while the reverse, with even the best of cements, leads to disappointment. For the construction of good concrete work, therefore, experience is the chief teacher—the art cannot be learned from books.

Experienced makers of hydraulic cement concrete will not be surprised at the strength and stiffness of the slender arches described in the paper. The immense strength of the egg-shell and of a continuous plate of any material, whether flat, bent or dished, indicates the possibilities of these forms when carefully made in cement mortar or concrete and thoroughly hardened. It is not at all easy, however, to get every one of a gang of workmen to follow out all day long every detail of the mixing, placing and protecting of fine concrete work. Some one or more will slight it somewhere at some time, and whenever this happens a weak spot occurs in the structure. The more successful the discipline, the more homogeneous and economical will be the product, and more uniform the strength. Thus, with constant vigilance, safe arches, domes, sewers and pavements of the slenderest forms and least quantities of material may be made. Whenever suitable machinery can be applied, uniform results are always assured, because it never becomes tired or careless.

The thinnest of arch rings, with spandrels filled, and the crown more or less covered, with even loose material, will carry great local loads through the distributing effect of the filling. The better the filling, the stiffer the arch, and so on. With strong cement mortar or fine concrete, containing wire or properly disposed light iron of some of the many new and extremely light sections now so readily and cheaply obtained in very long continuous lengths, selected and contrived intelligently for each new case of plate, arch or dome, almost wonderful results may be easily secured. Where we know what can be accomplished without the iron reinforcement, we may anticipate something very superior with it. In no case, however, may we abuse the cement at any point, from the time it is wetted until after the

expiration of the few weeks necessary for sound induration, without seriously impairing the structure. We must also look to the abutments and supports.

Hydraulic cement, fairly treated, is the engineer's friend, and, even like a faithful dog, not wholly alienated by abuse. The scope of its usefulness is still rapidly widening. In light brick and terra-cotta arching, as the matrix, it serves a most valuable purpose by converting the arch into a monolith like the continuous concrete arches described in the paper.

The so-called "Gustavino" arch, recently introduced in this country, is nothing more than the careful and intelligent construction of arch and dome shells with small rectangular 1-in. plates of hard terra-cotta, laid together on the flat-like tiles in either square or herring-bone bond. Two, three or four courses of these tiles are overlaid like the onion, breaking joints, with good Portland cement mortar in the joints and between the layers or concentric shells. When hardened the result is an extremely tough and stiff shell, capable of resisting very considerable local loads or shocks. A dome of this sort is like an egg shell on a large scale. Such construction, very useful and economical for many purposes, only illustrates what may be done with common bricks and cement mortar properly used. As a contribution to the published test of such arches, as well as of hollow terra-cotta work, much in the same line, I give the following, shown in Figs. 1 and 2, page 479.

Both arches were of hard red brick and Maryland cement mortar, 1 to 2, laid up in the regular course of construction without reference to special testing. "Cumberland" cement was used in the former, which was six months old at the time of the test, and "Round Top" cement in the latter, which was two years old.

The first arch (Fig. 1) was 34 ft. 5¾ ins. long, 10 ft. 5½ ins. span, and 11½ ins. rise. The haunches were one brick and the middle one-half brick thick, as shown, and the abutments were very heavy unyielding walls. One end of the arch was loaded close to the head, which was free, with a column of dry bricks, over the half-brick section of the arch ring, on an area of 7 ft. 6 ins. along the crown by 7 ft. transversely. About three-quarters of the load was applied during the first two days, and the remainder on the sixth day, when the test was stopped through fear that the column, then 13 ft. high, would topple over. The total load was 56 450 lbs., or 1 075 lbs. per square foot of loaded area. The deflection was $\frac{5}{16}$ in., at the crown, which entirely disappeared on removal of the load, and there was no crack or other sign of weakness.

The second arch (Fig. 2) was groined, springing from very heavy and unyielding abutments, between which the four heads were tightly enclosed, but not bonded thereto more than 3 ft. from the groin on the

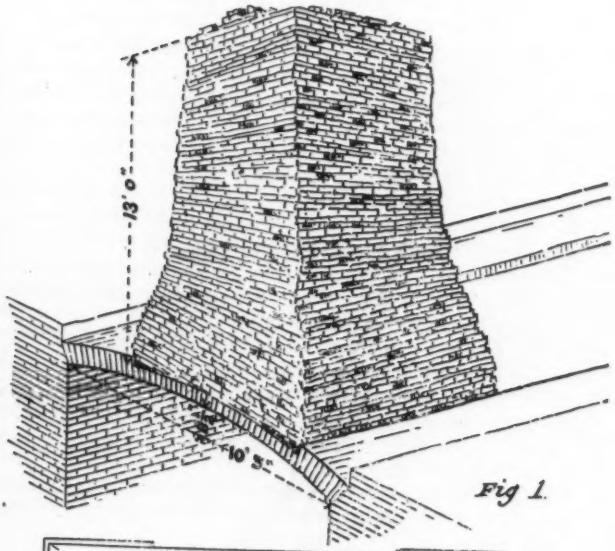


Fig 1.

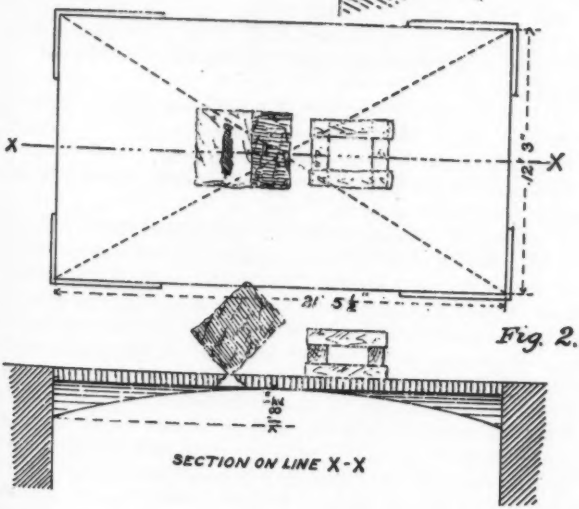


Fig. 2.

narrow head and 5 ft. on the wide head. These groined sections were 13 ins. thick, and all the remainder of the arch 8 ins., or one brick thick. The test of this arch consisted in tilting over on to the middle of its crown a nearly cubical block of granite, weighing 2¼ tons, from a blocking 2 ft. 3 ins. above the arch. The stone struck partially edge and corner wise, sending a heavy vibrating shock throughout the adjoining series of floor arches, of which this was one of five of about the same size, besides several other smaller arches. The sound and effect were similar to those of a heavy blow on a bass drum, and the only fracture or disturbance of any sort was a puncture of about 24 x 8 ins. splaying out below. The brick work spawled away like a monolith, quite regardless of joints or bond.

Fr. von Emperger, M. Austrian Soc. of Engrs. and Archts. (by letter).—I desire first to eliminate from this discussion such statements as cannot be found in my paper.

Messrs. Buck, Berger and Just claim that I have said that there will be no horizontal thrust in an arch. This is based on a statement regarding the Wünsch arch, page 444, beginning, "the inventor claims," etc., and which does not give my own opinion in this question. I avoided doing so, because I do not believe that it would have any value without further testing.

Mr. Wünsch's claims, however, may be true, inasmuch as a concrete beam, so shaped, may not act as an arch at all (see Plate LVII). I may further refer to an iron bridge built on this principle, the Stephanie bridge in Vienna. It is a cantilever bridge, with plate girders of an arch-like shape, and I understand that Mr. Buck and Mr. Berger refer to this point of view.

Mr. Just states that I have called "concrete inelastic like cast iron." On page 451, he will find that I quoted a technical paper as doing so, and stated that it is an unpardonable error.

Mr. Just quotes me as saying, "They will not have developed their full strength before three months."

This is an error in the same direction. The quotation is so far right, but not in the meaning given by this gentleman, that a concrete bridge could not be used before that time. The bridge will, after that time, be ten times as safe, and after three years, for instance, even more so. But this later increase will not be remarkable, and, therefore, the strength, after three months, may be taken as the standard. Every concrete bridge can be used immediately after completion, and centers will be struck as soon as it can be used without them, and the time for taking down centers varies from two months to two weeks (see page 452). Centers for floors are struck after one week, and I know of cases where this has been done after four days without bad results.

The time of striking centers must be determinated by the span, by the stress the dead load produces, by the kind of live load (railroad

bridges require the longest time), and by the extent to which the iron is able to carry the loads alone; as, for instance, in floors. For this reason the necessary time for setting will be longer with the Monier and Wünsch and shorter with the Melan system. The tremor from traffic has influence only during setting, and tests of cubes of concrete manufactured on a moving engine show a reduction of only 10% in strength, as compared with similar cubes which had set quietly. Mr. Just quotes me as saying that floors for buildings can be laid in this country without permits. On the last page of the paper, he will find that this had reference to highway bridges only.

So far as floors are concerned, all the building laws of the world, where there are any, have the same requirement as the New York law. Permits must be had for special or new constructions. But I hardly think that this can be taken as equal in value to a scientific testimonial like the permit for a bridge in Europe. It is only the opinion of an official who may not be competent, and depends a greal deal upon the personal influence of the promoter, or on other accessories. The statement given on page 456 applies only to Europe, and does not refer to this country.

In answer to Mr. Crowell, I may say that some recent tests of mortar (Meier-Malstatt), 1 to 6, did not show any remarkable change in expansion. Further, I would like to mention the tests of Dr. Früling, showing that the tensile strength of concrete is not affected by heat below red heat. This latter point is very important, but seems to me not sufficiently tested to allow of fair judgment.

Mr. Hutton's statements were of great interest to me. I have relied on unquestionable and well-known publications which he will have to correct. I may mention Trautwine, page 681. I am sorry that he has not mentioned his sources. The other errors mentioned are of less importance.

The bridge of 150-ft. span, near Ulm, is a foot bridge without hinges.

The remainder of the discussion I will divide into two heads, bridges and floors, and I intend, otherwise, to follow the statements of the authors as closely as possible.

Bridges.—Beginning with Mr. Gosling, I think he is correct, and my judgment of Coignet Béton should not have been given so abruptly.

The use of lime with Portland cement is, sometimes, so far as exterior finish is concerned, advisable with some kind of aggregates, but on account of a very regular abuse, I do not like it, and as there is no necessity for it, I prefer to be a little more particular about aggregates. For bridge work it cannot be recommended, and the composition called "Coignet Béton" does not keep volumes as well as Portland cement concrete.

Replying to Mr. O'Rourke, it seems well to state that a common concrete is not more able to stand the climate than any soft stone. If we remember how Nature has built sandstone and other aggregate stones, which is practically the same way as we build concrete, we know how to judge it.

The greater the pressure under which sandstone has sedimented, the less water remained, the less voids were left, the solider the stone, and the more fit is it to be used for exterior finish. Concrete should be judged in a similar manner. If we use the same concrete for exterior finish as we are accustomed to use for foundations or filling where much water and little ramming is necessary, we must fail.

Mr. O'Rourke gives an explanation of the cause of the shelling of the concrete, but I have strong doubts as to its correctness. If it really is possible, through change of temperature, to shell a concrete block, it would not be fireproof material at all, which, I think, nobody will doubt. The way this destruction of the surface occurs is by water which enters through the pores and freezes, as in any other stone. The shell will be just as thick as the frost can enter. This question is of even greater importance in harbor walls and with artificial stone of Portland cement, and I can say that it has been solved satisfactorily. It is only necessary to give the surface enough density not to allow the access of water, a similar process to that used in the construction of water-proof reservoirs and sewers of concrete. Grease, oil and some acids, which will be found not only in sewers, but also on the surface of the sea in harbors, are very detrimental to soft concrete. A dense surface will protect sufficiently, as the following test shows: Two cubes were prepared, the one of mortar, 1 to 3, and the other 1 to 1, and both were submerged in greasy oil. The first was destroyed, while the second hardened. I may refer to the perfect condition of a number of 20 year old oil tanks and sewers made of concrete.

There are many examples of the great damage which has been done by neglecting proper care of the exterior finish, especially in harbor work, where the worst weather and climate is joined with a great pressure of water, and, according to the tides, a continuous change of wet, dry and frost.

As an example of a failure could be mentioned the harbor of Aberdeen. Bamber & Carey on "Portland Cement," and Smith on "Portland Cement Concrete" (Minutes of Proceedings of the Institute of Civil Engineers, Session 1891–92), state that a lack of density of the surface is the only cause of the great damage which occurred there. All concrete work can be trusted to any reliable contractor, but to get out a good finish it needs something more than the experience of hearsay.

Much depends upon the choice of the aggregates as well as upon the amount of cement. Never use neat cement or a less mixture than

1 to 2½. The greatest density lies between 1 to 1 and 1 to 2½, and depends upon the quality of aggregates, which should be of all sizes, not dust-like, and the largest size in proportion to the thickness of the coating.

Regarding the inquiry of Mr. O'Rourke as to the amount of water used, I may say that, as a rule, concrete is strongest when made with the least amount of water. The amount can be proportioned in such a way that in ramming the concrete it shall show only a wet surface as a proof that enough water is in it.

A few words as to how to make an exterior finish.

There are two methods: First, to make common concrete and give it afterwards one or two coatings of dense mortar; and, second, to apply this coating inside while ramming the layers, and finish it up with a floating afterward. The method to be chosen depends upon location and other circumstances. It may be remarked, that, instead of a trowel, the use of a large wooden board is necessary, because it squeezes the water out equally and gives the coating an equal pressure. The coating must be well troweled in any case, so that the water rises to the surface, and floating and dusting continued until no more water appears. The second method, together with rubbing with a piece of felt, is generally used for artificial stone, and I may refer to the statue of Columbus at the World's Fair, and to the fountain, 20 years old, at the entrance of Prospect Park, Brooklyn, N. Y., as examples of the perfect finish which can be obtained.

The choice of material should not only be made by the eye, but always with sample tests.

An abstract of the report of Lieut.-Col. George L. Gillespie, Corps of Engineers, dated New York, N. Y., July 8th, 1893, is inserted with the permission of the author. He says:

"As a matter of professional interest, I desire to invite attention to the manner of constructing the counterscarp wall and galleries adopted by Lieut. Warren, to obtain a smooth surface, and one which is apparently weatherproof.

"The plan originally followed was to build all the concrete masonry in forms of undressed spruce lumber, and then, on removal of the forms, to plaster the exposed face with cement. This coat of plaster could not be made to adhere to the older masonry, which rapidly absorbed its moisture, weakening the bond between the two, which was destroyed by the first cold weather, when the coating cracked and dropped off, much to the injury of the appearance of the work.

"There was no economy in the use of spruce lumber, which warped out of shape when subjected to the moisture of the concrete, and could seldom be used more than once, and the plastering was very expensive.

"It was decided to substitute forms made of first quality dressed white pine, grooved on both edges, and united by loose tongues of yellow pine, and to construct the masonry as follows:

"That portion of the masonry next to the form, 4 ins. thick, was

put in without stone, the proportion of sand and cement remaining the same as in the concrete proper, securing a perfect bond between the face and the back of the wall.

"The forms being removed while the work was comparatively fresh, a little rubbing with a float gave a smooth surface to the face.

"The counterscarp wall and the galleries, constructed in this way, have withstood an unusually severe winter without scaling or cracking."

It is clear that this result was obtained by a little labor only, and that even better results are obtainable where the outside finish is more important.

But as in the Melan arch all iron is covered, I will not go deeper into this question. I have to state this definitely, because my designs do not show the coating which is supposed to be applied according to European practice, and are, therefore, somewhat misleading.

I may state further that English engineers in 1865, in building the West India Docks of London, thought that concrete could only be used as back filling, but they soon changed their minds, and in 1886 we read in the Minutes of Proceedings of the Institution of Civil Engineers, sessions 1886-87, a paper on "Concrete as Applied to the Construction of Harbors," that the use of every other material but concrete for harbor walls and dry docks belongs to the past.

I refer to the experience of the well-known firm of P. Ammann in Vienna, which has built, during 18 years' business experience, a very large number of water works, turbine dams and similar structures, and has always used only concrete walls with a concrete finish. If this was not satisfactory, they would certainly have changed their practice. Mr. P. Ammann states in a communication to me that he prefers a concrete finish on account of its low cost, as well as of its good quality,

My first impression was that Mr. O'Rourke's opinion was very common in this country, and consequently I intended to refer to much good concrete work which has stood for a long time in all weathers, but I soon desisted from so doing, especially as my attention has been called to the paper and discussion by Mr. Martin Murphy,* which covers the point.

Mr. Gayler asks whether an engineer is justified in reducing the dimensions to such dangerous limits. He will find in my paper that the safety maintained by those bridges is far above the safety usually given to iron bridges.

I do not feel inclined to discuss it further. I only desire to say that if an engineer would really use "his ingenuity to build the thinnest arch within the elastic limit," he would not deserve the name of engineer, but such an opinion should not be given without special proof. Whatever importance economical views may have, an engineer

* Trans. Am. Soc. C. E., Vol. xxix, p. 621, and Vol. xxx, p 567.

can only consider them after the engineering problem has been satisfactorily solved, as I have stated in my paper (page 454), giving also the economical question the last claim.

As this gentleman and Mr. Berger, as well as Mr. Hutton, express doubts, which on account of the space I have not answered fully, I submitted my paper to Prof. William H. Burr, of Columbia College, having been advised that he is one of the best authorities in this country regarding the theory of arch ribs.

The following is his statement:

"I have carefully examined the theory of the continuous arched rib with fixed ends, both under and independent of that given by Prof. Melan, on which the formulas of your paper on the Melan System of Arch Construction presented before the American Society of Civil Engineers, April 4th, 1894, are based.

"I find that those formulas are developed from correct theory, and, hence, that arches designed under them, with proper values of the empirical quantities which enter them, will be correctly proportioned and entitled to every confidence usually given to rationally designed engineering structures."

For further explanation it seems well to state that it is not an approved practice, as some of the gentlemen understood from my paper, to use concrete alone in tension. The calculations of concrete arches in Germany are generally based on the assumption that they are not subjected to tension by the safe live loads. The view taken in regard to the safety, and, consequently, to the breaking, load of such arches—loads which never occur, but are a scale on the reliability of the material—is quite another matter.

In a masonry arch the cohesion of mortar joints is taken as superfluous, and every part where tension occurs is practically supposed not to be a part of the arch. A stone arch must be a safe structure, with no mortar at all. Tension is taken into consideration for breaking loads, and the necessary safety can be easily attained.

I may refer to the tests made by Professors Engesser, Dyckerhoff, Pittel and Brautewetter on concrete arches alone.

It is good practice, and has been for a number of years, to allow tension in concrete when used in combination with iron.

The limit proposed by Prof. Melan, 60 lbs. per square inch, is the lowest to my knowledge, and according to the proportion of the elastic moduli the iron will have a tension of 2 400 lbs. per square inch. The concrete is not strained in two directions under maximum stresses, but for that case the stress resulting from both stresses is reduced to a lower limit of 42 lbs. per square inch. The spacing of beams is based on this supposition (see equation No. 3). While in the case of a concrete arch any improper connection in concrete will necessarily continue through the neighboring concrete, in this case it cannot show before the tensile limit of the iron rod keeping the concrete tight together is attained; that is, about 30 000 lbs. per square inch. We see

that the iron in this direction has a headroom of 13 in proportion to the stress which it is designed for. It is practically the same as if we had a stone arch connected at the joints by ties.

The omission of the action of the temperature was made in regard to the space which it would require and in regard to the fact that it does not need to be cared for in the spans of the length considered. For spans over 150 ft., hinges are to be recommended of the style Mr. Triest has described, and for the reasons given by Mr. Hutton.

The arch and the railing expand differently, and, if not specially designed and expansion joints provided, the latter will be cracked, as can be seen in every larger stone arch. I agree with the opinion of Mr. Hutton that this way of construction may have a great future and that it is superior to the usual arch with fixed ribs for larger spans.

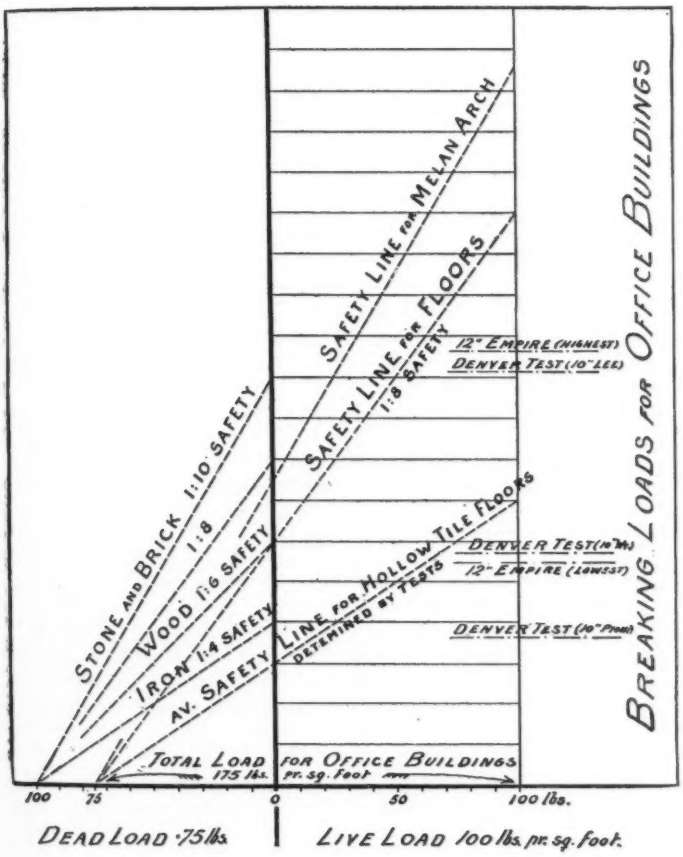
Mr. Gayler further says, regarding the Monier system, that "it will probably never be used for spans exceeding 50 ft., as it admits in no wise of a calculation of its strength." This is not well founded, as quite a number of Monier bridges have a larger span than 100 ft., and if he does not know how to calculate them, that is not a proof that we have not a quite reliable theory which is perfectly in accord with the tests. I have omitted the calculation regarding Monier and Wünsch, to reduce the size of my paper. I should think that tests of arches of 14-ft. and 77-ft. span, as made by the Austrian Society of Engineers and Architects, should cover the ground, especially in comparing this theory with those of iron bridges, the latter not always being backed by such full-sized tests. The specimens in all cases were made expressly for testing. Regarding the time, it is a fact that concrete improves, while this cannot be said of iron. A period of 10 years should be taken as a sufficient proof for an assertion of that kind.

The railroad crossing at Moedling has as heavy a traffic to carry as any other road. The dead load of concrete arches (as shown in Plate LXI) is 450 lbs. per square foot; the dead load of an iron bridge (Plate LX) 50 lbs. per square foot. Compared with the live load of 100 lbs., this gives a proportion of 9: 1: 2. It is consequently not bold, in my opinion, to claim that the former is free from vibration and tornado-proof, and the more so, as the connection with the supports is a far solider one than with iron bridges.

Regarding the tensile strength of concrete, I think it well to state that the Austrian Society of Engineers and Architects' standard does not call anything Portland cement, which gives in the proportion of 1: 3 after 28 days a tensile strength less than 200 lbs. per square inch, as an average of four samples. Quite a number of cement firms are willing to guarantee even a higher limit.

The strength of Portland cement concrete is equal to the tensile strength of the mortar, the cohesion between stone and cement being the same. But there is no reason to go to so high a guarantee if we want a concrete of 310 lbs. per square inch after three months. I think a

PLATE LXII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 703.
VON EMPERGER ON CONCRETE-IRON BRIDGES.


BREAKING LOADS FOR OFFICE BUILDINGS
SAFETY LINE FOR MELAN ARCH
SAFETY LINE FOR FLOORS 1:8 SAFETY
12" EMPIRE (HIGHEST)
DENVER TEST (10" LEE)
STONE AND BRICK 1:10 SAFETY
WOOD 1:6 SAFETY
IRON 1:4 SAFETY
AV. SAFETY LINE FOR HOLLOW TILE FLOORS DETERMINED BY TESTS
DENVER TEST (10" M.)
12" EMPIRE (LOWEST)
DENVER TEST (10" Prout)
TOTAL LOAD 175 lbs. FOR OFFICE BUILDINGS pr. sq. foot
100
75
0
50
100 lbs.
DEAD LOAD 75 lbs.
LIVE LOAD 100 lbs. pr. sq. foot.

guarantee of 150 lbs. sufficient, but would advise, in considering bids, not to consider the price only, but also the guarantee.

Wherever transverse and tensile tests of mortar are made together, their connection has been clearly proven.

Floors.—I will finally, in a few lines, touch upon iron concrete for floor construction.

A lively interest has been taken in its possible use for floors in fireproof buildings. This was unexpected, as my paper was intended, principally, to consider concrete iron as a bridge-building material. On this account, believing that the main features of the combination have been dealt with, I will confine myself only to an analysis of the comparison of a hollow tile floor and a Melan arch floor, as given by Mr. George Hill.

The diagram (Plate LXII) shows the present state of the floor question. I give only the figures of tests, which are, in my opinion, reliable as not being made by the manufacturer himself.

I. Tests with 12-in. block of Empire Fireproofing Company were made in 1891 by Mr. Cutshaw, City Engineer, Richmond, Va. They show a variation of the breaking load from 554 to 1 057 lbs. per square foot.

II. The well-known Denver tests, 1892, with 10-in. blocks of different make, showing a breaking load of 360 to 571 lbs. and 1 008 lbs., respectively.

From the diagram it will be seen that with my proposed reduction of safety to 1 : 8 an office building with 175 total load requires a breaking load of 1 400 lbs. per square foot, and as long as no tests with hollow tile can be produced showing these figures, Mr. Hill has no right to compare them with a Melan floor or any other floor (for instance, a wooden floor) which is constructed on correct engineering principles.

We meet people who are quite satisfied with a floor as long as it does not fall down, and I myself have seen hollow tile floors which could hardly have had a factor of safety higher than 2. It is certainly not sound engineering to advocate a method of construction on account of its cost only.

The following test was made last year at Brünn, Austria :

Three beams supported two 7-ft. spans, the two outside beams connected by heavy tie-rods and the middle beam left free to move. One span was then loaded with 1 400 lbs. per square foot for the whole surface. There were no signs of overloading, and the middle beam yielded only $\frac{1}{8}$ of an inch, and recovered afterwards when the load was removed, although free to move and without horizontal bracing. The distance between the rods in a common brick arch should not be more than 5 ft., and without tie-rods it would fall down at its earliest convenience. This is not the case with the Melan system. A proper factor of safety is necessary to the long life of a building. The Roman

concrete bridges could not have lasted 600 years if they had been so closely calculated to their load. Our iron bridges with a safety of 5 are not supposed to live more than two generations. What would happen if these figures are lowered further? Certainly our most splendid buildings could in that case not be called permanent.

Mr. Hill gives us a striking simplification of the method by which a Melan arch should be calculated, which is based upon his opinion that an arch can be calculated in the same way as a tie-rod. He considers the horizontal thrusts only. This leads him to say that the tension member (tie-rod) should have the same section as the arch itself. See his comparative table and his statement that "it will have the same amount of metal required as in a beam minus the web."

There is hardly any necessity to go further into Mr. Hill's theories. The same calculation of a hollow tile floor gives him a safe load of 770 to 1 027 lbs. per square foot. This means a breaking load of 7 700 to 10 270 lbs. per square foot, as he has used a factor of safety of 10. There are few tests which show a higher breaking load than his safe load, and this alone should warn us that there is something wrong in it.

I have given my opinion on this subject in a series of articles in the *Engineering Record*, under the heading "Testing Hollow Tile Floors," March 3d–31st, which also contains comparative designs for the table below.

The prices as given in Mr. Hill's table for the hollow tile floor I believe to be fair for New York prices, except that 5 cents per ½-in. leveling seems disproportionately large, but $18 90 per cubic yard for concrete, and $12 15 for leveling (ashes), in the Melan arch, must seem to anyone exorbitant prices, even for New York City.

Material.	Hollow Tile Floor.	Melan Floor.	
		6-ft. 8-in. span.	20-ft. span.
Beams and girders	10.1 lbs.	9.2 lbs.	4.0 lbs.
Connections	1.04 "	0.92 "	.....
Tie-rods	0.3 "	.....	0.95 "
	Cents.	Cents.	Cents.
	11.44 lbs. at 3 cts .. 34.3	10.12 lbs. at 3 cts...30.4	4.95 lbs. at 3 cts. 14.9
Bent shapes	.....	1.8 at 3½ 6.3	2.25 at 3½ cts. 7.9
Arching	 26	.....14.	20
Leveling	 2	..... 4.	 7
Cost of Construction per square foot	..... 62.3	.....54.7	.....49.3
Depth	16 ins. at 2 cents... 32	16 ins. at 2 cents.. .32.	13 at 2 cents. ...26
	94.3	86.7	75.9
		Flat ceiling 6.0	
		92.7	